The 2007 International Forum on Landslide Disaster Management

Edited by:

Ken Ho
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Victor Li
Victor Li & Associates Ltd., Hong Kong

Volume I
Cover photographs:
The 1990 Tsing Shan channelised debris flow in Hong Kong (left) and three-dimensional landslide mobility modelling (right).

The copyright is owned by the authors. No part of this publication or the information contained herein may be reproduced, stored in a retrieval system, or transmitted in any form or by any means, electronic, mechanical, by photocopying, recording or otherwise, without prior written permission from the authors.

Although all care is taken to ensure the integrity and quality of this publication and the information herein, no responsibility is assumed by the publishers nor the authors for any damage to property or persons as a result of use of this publication and/or the information contained herein.

Published by:
Geotechnical Division, The Hong Kong Institution of Engineers


Printed in Hong Kong
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foreword</td>
<td>VII</td>
</tr>
<tr>
<td>Organisation</td>
<td>IX</td>
</tr>
<tr>
<td>Sponsors</td>
<td>XI</td>
</tr>
<tr>
<td>Technical Programme</td>
<td>XIII</td>
</tr>
</tbody>
</table>

## VOLUME I

Opening Address

*R.K.S. Chan*  
1

**Session 1  Country / Regional Reports on Landslide Risk Management Practice**

Development of slope management in Malaysia

*C.H. Abdullah, A. Mohamad, M.A.M. Yusof, S.S. Gue & M. Mahmud*  
3

Landslide risk management in Hong Kong

*R.K.S. Chan & S.H. Mak*  
17

Experience on landslide risk management in the Eastern Pyrenees (Spain and Andorra): Achievements and challenges

*J. Corominas*  
49

The Alerta Rio System

*R.N. d’Orsi*  
71

Report on landslide impacts and practices in Switzerland: Need for new risk assessment strategies

*M. Jaboyedoff & Ch. Bonnard*  
79

Landslide risk management: Country Report for Norway

*S. Lacasse & F. Nadim*  
99

Slope safety and landslide risk management in Korea

*S.G. Lee & S.R. Hencher*  
125

Landslide risk management in France: Principles, organisation and challenges

*E. Leroi*  
169


*A. Leventhal*  
205

Evolution of slope-land hazard mitigation strategies and measures in Taiwan

*M.L. Lin & H.L. Wu*  
235

Landslide hazard activities in the United States

*P.T. Lyttle*  
251
Challenges in landslide risk management in a European perspective  
*D. Patel & O. Kjekstad*  
Page 261

Landslide disaster management in Italy  
*L. Picarelli, P. Versace, R. de Riso & M. Palmieri*  
Page 281

Country Report from Japan: Progress of landslide dynamics and the International Programme on Landslides  
*K. Sassa*  
Page 319

Landslide risk management in the United Kingdom  
*M.G. Winter, R.G. McInnes & E.N. Bromhead*  
Page 343

Landslide mitigation strategy and implementation in China  
*Y.P. Yin, S.J. Wang & Z.Y. Chen*  
Page 375

Session Chair’s report on landslide risk management practice  
*A.W. Malone*  
Page 387

Record of discussion  
*J.C.Y. Cheuk, H.W. Sun & J.W.C. Lau*  
Page 391

**Session 2  Forensic Landslide Investigations**

The investigation of the Aznalcóllar dam slide failure  
*A. Gens & E. Alonso*  
Page 409

Reflections on forensic investigations to determine the nature, risk, and causes of the 1979 Abbotsford landslide, Dunedin, New Zealand  
*G.T. Hancox*  
Page 429

Repeated collapse of cut slopes despite remedial works  
*S.G. Lee & S.R. Hencher*  
Page 451

The Storegga slide - case study of an offshore megaslide in the Norwegian Sea  
*F. Nadim & T.J. Kvalstad*  
Page 465

Landmark landslides in Malaysia  
*M.A. Othman, M. Mahmud, A. Mohamad & M.J. Sulaiman*  
Page 485

Landslide investigations and risk mitigation: The Sarno case  
*P. Versace, G. Capparelli & L. Picarelli*  
Page 509

Insights from some milestone landslides in Hong Kong  
*H.N. Wong & K.K.S. Ho*  
Page 535

Landslide hazard monitoring and warning system for Li-shan area  
*H.L. Wu & M.B. Su*  
Page 555

Session Chair’s report on forensic landslide investigations  
*L. Picarelli*  
Page 567

Record of discussion  
*J.S.H. Kwan & H.W. Sun*  
Page 571
Session 3  Innovative and Digital Technology

Predicting slope failure using real-time monitoring technology and the TRS sensor
K.T. Chang, H.S. Han, J.F. Wang, A.N.L. Ho & D. Mothersille 579

From permanent scatterers to WebGIS: Innovative techniques for landslide monitoring and early warning
G. Falorni, P. Canuti, P. Farina, L. Leoni & N. Casagli 587

Assessment of slope sensor data to support rock slope stability analysis and infrastructure hazard management
D.J. Hutchinson, M. Diederichs, K. Kalenchuk & M. Lato 599

Development and application of geoinformatics for landslide risk management in Hong Kong
K.C. Ng & H.N. Wong 619

Prediction of rainfall-induced landslides in unsaturated granular soils for setting up of early warning systems
L. Picarelli, P. Versace, L. Olivares & E. Damiano 643

Application of time domain reflectometry for quality control of soil nailing works

Advances in real-time monitoring of slope stability
J.M. Strout, E. DiBiagio & R.G. Omli 687

Development of a method for multi-scale landslide risk assessment in Cuba
C.J. van Westen & E.A. Castellanos Abella 717

Session Chair’s report on innovative and digital technology
S. Lacasse 747

VOLUME II

Session 4  Benchmarking Exercise on Landslide Debris Runout and Mobility Modelling

Review of benchmarking exercise on landslide debris runout and mobility modelling
O. Hungr, N.R. Morgenstern & H.N. Wong 755

The 2005 Tate’s Cairn debris flow: Back-analysis, forward predictions and a sensitivity analysis
J. Cepeda 813

Analysis of Hong Kong debris flow with an energy based model
D. Chan, N.R. Morgenstern, D. Tran & X.B. Wang 835

Landslide mobility analysis using MADflow
J.H. Chen & C.F. Lee 857

Approach to numerical modelling of long runout landslides
G.B. Crosta, S. Imposimato & D. Roddeman 875
Benchmarking TITAN2D mass flow model against a sand flow experiment and the 1903 Frank slide
*S. Galas, K. Dalbey, D. Kumar, A. Patra & M. Sheridan*

Two models for analysis of landslide motion: Application to the 2007 Hong Kong benchmarking exercises
*O. Hungr, M. McKinnon & S. McDougall*

Application of 2D-finite volume code FLATModel to landslide runout benchmarking exercises
*M. Hürlimann, V. Medina & A. Bateman*

Benchmarking exercise on landslide mobility modelling - runout analyses using 3dDMM
*J.S.H. Kwan & H.W. Sun*

Benchmark exercises for granular flows
*A. Lucas, A. Mangeney, F. Bouchut, M.-O. Bristeau & D. Mège*

A SPH depth integrated model with pore pressure coupling for fast landslides and related phenomena
*M. Pastor, T. Blanc, M.J. Pastor, M. Sánchez, B. Haddad, P. Mira, J.A. Fernández Merodo, I. Herreros & V. Drempetic*

A set of benchmark tests to assess the performance of a continuum mechanics depth-integrated model
*M. Pirulli & C. Scavia*

Landslide detachment mechanisms: An overview of their mechanical models
*R. Poisel & A. Preh*

Punta Thurwieser rock avalanche and Frank slide: A comparison based on PFC$^{3D}$ runout models
*R. Poisel, A. Preh & O. Koc*

Landslide simulation by geotechnical model adopting a model for variable apparent friction coefficient
*F.W. Wang & K. Sassa*

Report on benchmarking exercise of landslide debris runout and mobility modelling
*J.S.H. Kwan & H.W. Sun*

Closing Address
*L. Picarelli*

Photographs

Register of Participants

Author Index
FOREWORD

The International Forum on Landslide Disaster Management was held in Hong Kong in December 2007 to commemorate the 30th anniversary of the implementation of landslide risk management by the Geotechnical Engineering Office, Government of the Hong Kong Special Administrative Region (SAR). The event was convened under the auspices of the Joint Technical Committee on Landslides and Engineered Slopes (JTC-1) of the ISSMGE, ISRM and IAEG.

The Forum has gathered together some 60 participants, who have all been invited by virtue of their expertise in slope engineering, to share their experience and insights. The papers contained in this set of Proceedings have been contributed by eminent international landslide experts. Also included in the proceedings is a record of the discussions and speeches made. The accompanying CD contains the papers as well as the presentation materials.

The papers are arranged under 4 sessions. The first session comprises country or regional reports on landslide risk management practice. The second session covers forensic landslide investigations whilst the third session is on the use of innovative and digital technology in slope engineering. The last session consists of a benchmarking exercise on landslide mobility modelling, which was steered by a Review Committee comprising Norbert Morgenstern, Oldrich Hungr and HN Wong.

The Forum was jointly organised by the Geotechnical Division of the Hong Kong Institution of Engineers, the Geotechnical Engineering Office of the Government of the Hong Kong SAR and the Hong Kong Geotechnical Society. The Organising Committee wishes to express its gratitude to these organisations and to the sponsors named in the volume.

Ken Ho  
Chairman, Organising Committee  
Hong Kong  

October 2008
ORGANISATION

Organising Committee

Chairman
Ken Ho

Members
Johnny Cheuk
Albert Ho
May Ho
Y C Koo
David Kwok
Rachel Law
Alex Li
Eric Li
Victor Li
L M Mak
Charles Ng
Loretta Pau
W K Pun
H W Sun
George Tham
Jenny Yeung
J H Yin
Ringo Yu

International Advisory Committee

Robin Fell
Suzanne Lacasse
Willy Lacerda
C F Lee
Norbert Morgenstern
Luciano Picarelli

Review Committee for Benchmarking Exercise on Landslide Runout Analysis

Oldrich Hungr
Norbert Morgenstern
H N Wong
SPONSORS

China Geo-Engineering Corporation
C M Wong & Associates Ltd.
Fraser Construction Company Ltd.
Fugro (Hong Kong) Ltd.
Fuk Shing Engineering Company Ltd.
Halcrow China Ltd.
LMM Consulting Engineers Ltd.
Maunsell Geotechnical Services Ltd.
New Concepts Engineering Development Ltd.
Ove Arup & Partners Hong Kong Ltd.
Tysan Foundation Ltd.
Victor Li & Associates Ltd.
TECHNICAL PROGRAMME

Monday, 10 December 2007

08:30 Opening Address
R.K.S. Chan

Session 1 Country / Regional Reports on Landslide Risk Management Practice
(Chair: A.W. Malone)

A. Leventhal

Landslide Mitigation Strategy and Implementation in China
Y.P. Yin

Landslide Risk Management in Hong Kong
R.K.S. Chan

Evolution of Slope-land Hazard Mitigation Strategies and Measures in Taiwan
M.L. Lin

Country Report from Japan: Progress of Landslide Dynamics & the International Programme on Landslides
K. Sassa

10:30 Coffee Break

10:50 Slope Safety and Landslide Risk Management in Korea
S.G. Lee

Development of Slope Management in Malaysia
A. Mohamad

Landslide Risk Management: Country Report – Canada
J. Hutchinson

11:50 Panel Discussion

12:30 Lunch

14:00 Landslide Hazard Activities in the United States
P.T. Lyttle

Challenges in Landslide Risk Management in a European Perspective
F. Nadim

Landslide Risk Management in France: Principles, Organisation and Challenges
E. Leroi

Landslide Disaster Management in Italy
L. Picarelli

Landslide Risk Management: Country Report for Norway
S. Lacasse
15:40 Coffee Break

16:10 Experience on Landslide Risk Management in the Eastern Pyrenees (Spain and Andorra): Achievements and Challenges  
_J. Corominas_

_M. Jaboyedoff_

Landslide Risk Management in the United Kingdom  
_M.G. Winter & R.G. McInnes_

17:10 Panel Discussion

18:30 End of Session

**Tuesday, 11 December 2007**

**Session 2  Forensic Landslide Investigations**  
(Chair: L. Picarelli)

08:30 The Investigation of the Aznalcóllar Dam Slide Failure  
_A. Gens_

The Storegga Slide – Case Study of an Offshore Megaslide in the Norwegian Sea  
_F. Nadim_

Landslide Hazard Monitoring and Warning System for Li-shan Area  
_M.B. Su_

Repeated Collapse of Cut Slopes Despite Remedial Works  
_S.G. Lee_

09:55 Panel Discussion

10:30 Coffee Break

10:50 Landmark Landslides in Malaysia  
_A. Othman_

Landslide Investigations and Risk Mitigation: The Sarno Case  
_P. Versace_

Insights from Some Milestone Landslides in Hong Kong  
_H.N. Wong_

11:55 Panel Discussion

12:30 Lunch
Session 3  Innovative and Digital Technology  
(Chair: S. Lacasse)  
14:00  Assessment of Slope Sensor Data to Support Rock Slope Stability Analysis and Infrastructure Hazard Management  
J. Hutchinson  
Advances in Real-time Monitoring of Slope Stability  
R.G. Omli  
Development and Application of Geoinformatics for Landslide Risk Management in Hong Kong  
K.C. Ng  
Predicting Slope Failure Using Real-time Monitoring Technology and the TRS Sensor  
K.T. Chang  
Application of Time Domain Reflectometry for Quality Control of Soil Nailing Works  
W.K. Pun  
From Permanent Scatterers to WebGIS: Innovative Techniques for Landslide Monitoring and Early Warning  
G. Falorni

15:50  Coffee Break

16:10  Prediction of Rainfall-induced Landslides in Unsaturated Granular Soils for Setting Up of Early Warning Systems  
L. Picarelli  
Development of a Method for Multi-scale Landslide Risk Assessment in Cuba  
C.J. van Westen

16:40  Group Discussion

17:30  Plenum Discussion

18:30  End of Session

Wednesday, 12 December 2007

Session 4  Benchmarking Exercise on Landslide Debris Runout and Mobility Modelling  
(Chairs: O. Hungr, H.N. Wong & H.W. Sun)  
08:30  Overview on Benchmarking Exercise  
H.N. Wong, H.W. Sun & O. Hungr

09:00  Benchmarking Exercise – Presentations  
J.H. Chen  
O. Hungr  
M. Pastor  
J.S.H. Kwan
M. Pirulli
M. Sheridan

10:30 Discussion
10:50 Coffee Break
11:10 Benchmarking Exercise – Presentations
   K. Sassa & F.W. Wang
   A. Lucas
   J. Cepeda
   D. Chan
   R. Poisel
   G. Crosta
   H.W. Sun (on behalf of M. Hürlimann)
12:40 Discussion
13:00 Lunch cum Exhibition on Digital Technology
15:00 Summary of Benchmarking Exercise Results
   O. Hungr & H.N. Wong
15:30 Discussion
16:10 Closing Address
   L. Picarelli
16:30 Exhibition on Digital Technology
17:30 End of Programme
Opening Address
OPENING ADDRESS

Ladies and gentlemen, good morning. For those who come from overseas, welcome you all to Hong Kong!

Landslide is the most common form of natural disaster in Hong Kong given its densely populated development in the urban areas close to our steep hillsides and also with our high seasonal rainfall. Some 35 years ago, a landslide disaster took place on the very hillside behind where we are now today. A 12-storey high-rise building was struck and completely demolished by a massive landslide, killing 67 people and injuring 20 persons. I was then studying in the civil engineering department of this university. Although this disaster occurred some 35 years ago, I still remember the incident vividly.

A series of landslide disasters with multiple fatalities in the 1970s led to strong public outcry and culminated in the establishment of the Geotechnical Control Office in 1977 (we are now known as the Geotechnical Engineering Office, in short, GEO). One of our main duties is to regulate slope safety and our mission is to save lives. A comprehensive slope safety management system has been formulated, and has continued to evolve over the years in response to lessons learnt. This year is the 30th anniversary of landslide risk management in Hong Kong, and it is opportune to have so many international experts gather together here to exchange experience on the subject.

Although the overall landslide risk to the local community has been reduced considerably, there is no room for complacency. As public safety is at stake, the Hong Kong SAR Government is committed to meeting the needs of Hong Kong for the highest standards of slope safety. As such, we will continue to strive to use the best practices in landslide risk management and the state-of-the-art knowledge, technology and tools in combating landslide threat.

On behalf of the geotechnical profession in Hong Kong, it is my pleasure to welcome you to the International Forum on Landslide Disaster Management, which is organized by the Geotechnical Engineering Office of the Hong Kong SAR Government, the Hong Kong Institution of Engineers and the Hong Kong Geotechnical Society, under the auspices of the Joint Technical Committee JTC-1 of the ISSMGE, IAEG and ISRM.

I am sure we will all benefit from the sharing of insights in the course of this 3-day programme. Enjoy the interaction and the knowledge, and wish you all a fruitful and a productive forum.

Thank you very much.

R.K.S. Chan
Head of the Geotechnical Engineering Office
Session 1  Country / Regional Reports on Landslide Risk Management Practice
DEVELOPMENT OF SLOPE MANAGEMENT IN MALAYSIA

C. H. Abdullah, A. Mohamad and M. A. M. Yusof
Slope Engineering Branch, Public Works Department, Kuala Lumpur, Malaysia

S. S. Gue
G & P Professionals Sdn Bhd, Kuala Lumpur, Malaysia

M. Mahmud
Kupulan Ikram Sdn Bhd, Selangor, Malaysia

Abstract: Malaysia has her fair share of slope problems in tandem with the increase in development that encroaches into the hilly areas. The first landmark landslide event occurred in 1993 when a condominium block constructed on a slope toppled over due to a landslide that undermined the structure. There were other landslide events that occurred subsequent to 1993 event that resulted in many fatalities and more destruction but no concrete action was carried out to manage the slope. However, the event has created awareness amongst the public. After a series of disastrous high profile landslides following the 1993 landslide event, the Malaysian government established a Slope Engineering Branch within the Public Works Department of Malaysia in 2004. One of the first missions assigned to the Slope Engineering Division is to develop a slope master plan for the whole of Malaysia. The Slope Master Plan is a blueprint for the management of slopes and other matters related to slopes which include emergency preparedness, early warning system and public awareness. This paper describes the slope development in Malaysia that has transpired since the first landmark landslide event in 1993.

INTRODUCTION

Malaysia is located between 1 and 6 degrees north of equator. Our neighbors are Thailand and the Philippines in the north, Singapore in the south and Indonesia in the east and west. Due to its location, Malaysia has a tropical climate, with high-intensity and frequent rainfall, and high humidity. The temperature in the lowlands varies between 24°C and 33°C throughout the year. Two monsoon seasons from the north-east and the south-west bring plenty of rain to Malaysia. The population is approximately 27 millions (Department of Statistics 2007) which consists mainly of Malay, Chinese, Indian, and indigenous groups in Sabah and Sarawak. Malaysia is divided into three regions: Peninsular Malaysia located in the west; Sabah and Sarawak located in the island of Borneo in the east. The total land area of Malaysia is 330,000 km². The average rainfall is approximately 2600 mm per annum although the highest rainfall of nearly 6000 mm has been recorded in Sandakan, Sabah. Hourly rainfall of approximately 160 mm has also been recorded in the same area (Malaysia Meteorological Department 2007).

Due to high-intensity and total rainfall, Malaysia experienced frequent landslides, mostly rainfall induced. Records obtained from newspaper reports since 1966 indicate that before 1993, there were no fatalities due to landslides. This may be partly because of development during those periods occurring near rivers and in flood plain areas which were located at a distance away from the hilly areas. Only in late 1980s, extensive developments began to encroach into hilly parts of the towns and cities. Landslide did not come into prominence
until 1993, when a landslide undermined the foundation of a condominium block that caused it to collapse - killing 48 people. This paper presents the development of slope management since the first landmark landslide in 1993.

BACKGROUND

General Geology of Malaysia
Geology plays an important role in slope stability especially when it involves rocks, rock weathering, and residual soils of different nature and properties. Understanding the geology will in some ways help in understanding the possible mechanism of failures. For example, Sabah is prone to landslide activities due in parts to its active past tectonic activities that generated discontinuities in the rocks and extensive weathering as a result of these discontinuities. Geographically, Malaysia is divided into three regions, the Peninsular Malaysia, Sabah and Sarawak. The geology of these three regions varies distinctively from one another and from simple geology in the Peninsular Malaysia to complex geology in Sabah and Sarawak.

The Peninsular Malaysia forms part of the stable Sunda Shield. All the systems, ranging from the Cambrian to the Quaternary, are represented in Peninsular Malaysia. In the Peninsular Malaysia, the oldest rock is a Cambrian rock comprised of predominantly arenaceous in character. This was followed by the deposition of thick sequence of limestone, carbonaceous argillaceous rocks (phyllites, shale, and slate), and arenaceous rocks. The deposition occurred under different environments from Silurian to Carboniferous. Due to various orogenies, unconformities can be found between the rock successions. Granites which occupy about 50% of the total surface area of Peninsular Malaysia, emplaced through at least after four major episodes of orogenies, which occurred during the Upper Carboniferous to Lower Permian times. Large intrusion of granitic rock occurred during large orogeny in the Upper Permian until Lower Triassic. The Upper Triassic orogeny has divided the Peninsular Malaysia into north-south physiographic trend and is aligned parallel to the structural trend of the country. The granite bodies commonly form topographic highs, the largest of which is the Main Range Granite with the length of about 480 km and an average width of 65 to 80 km and rising to more than 2100 m above the sea level in places. Regional metamorphism is widespread which shows slight deformation and is commonly associated with faulting. At least three sets of faults have been recognized on regional scale. Figure 1 presents the simplified geological diagram of Peninsular Malaysia.
Figure 1: Simplified geology of Peninsular Malaysia

The geology of Sarawak is generally recognized as belonging to two distinct provinces, namely West Sarawak and Central-North Sarawak. In West Sarawak, the oldest rocks are of pre-Late Carboniferous age. Successive sedimentary formations in this province are the rocks of Carbon-Permian, Triassic, Jura-Cretaceous, and Upper Cretaceous - lower Tertiary times. Four major unconformities are apparent in West Sarawak and they represent the major breaks in the sedimentary records. In Central – North Sarawak, the oldest rocks known are of Cretaceous age, occurring in the southwest margin of the province. The rest of the province is underlain by Tertiary rocks with some isolated basins of Quaternary rocks. The Late Eocene unconformity marks the major break in the sedimentary record in this province. The igneous rocks of Sarawak can be broadly classified into three major phases of igneous activities. The Palaeozoic - Triassic and the Jurassic - Cretaceous phases are confined strictly to West Sarawak whereas the main Late Cretaceous – Quaternary phase of igneous activities took place in Central – North Sarawak, and to lesser extent, in West Sarawak. Regional metamorphism is confined to the pre-Upper Carboniferous rocks and some of the Jurassic-Cretaceous rocks in West Sarawak, and rocks older than Late Eocene in Central – North Sarawak. Thermal metamorphism is limited to narrow aureoles around some of the igneous
bodies. Faulting and folding are common and appear to have affected all the rocks except the Quaternary. Figure 2 shows the geology of Sarawak. Sarawak however is less mountainous than Sabah, she experienced less devastating landslides than Sabah and Peninsular Malaysia.

Sabah, which is situated at the northern tip of Borneo Island, due to active tectonics from surrounding regions, shows a complex geology. The Sulu Volcanic arc which came ashore at the east coast provided much of the volcanic and volcanoclastic materials to a whole east coast during Tertiary time while the Sundaland Craton to the southwest provided large amounts of clastics sediments to a marine trough which later formed the Crocker Ranges. A diverse group of rocks of metamorphicoceanic lithospheric origin, well-bedded deep water calstis sediments, ophiolites suites to chaotic melanges, volcano-sedimentary sequences and shallow basin sediments were evolved from the effect of subduction, obduction, arc associated tectonism and volcanism in and around Sabah. The structural geology of Sabah is dominated by two sets of faults; a north-northeast set and a northwest to north-northwest set around Gunung Kinabalu. A large strike-slip fault, the ‘Kinabalu fault’ cut across Sabah from the northwest coast through Gunung Kinabalu and the Labuk Valley to the southeast coast between Cowie Harbour and Darvel Bay. Various intensity of folding, from concentric fold to isoclinal, can be seen in the sedimentary rocks. Igneous rocks in Sabah are varied in composition and origin. The geology of Sabah is presented in Figure 3.

Economic Development and Landslides
Malaysia experienced rapid economic growth in the 1990s until the end of 1997, when there was an economic crisis in Asia resulting in a major economic downturn in Malaysia. The
economic growth of Malaysia between 1990 and 1997 was more than 7% per annum. There was a sharp drop in the economic growth at the end of 1997 and 1998 before rebounding again in 1999. For the past 7 years since 2000, the Gross Domestic Product (GDP) has grown by more than 5%, while the growth in the construction sector has become slightly sluggish with an average growth of 1%. In the same period, the yearly construction sector output is averaging slightly above 2 billion USD. The construction sector contributed approximately 3% of the GDP. As a result of the economic growth, the construction sector in Malaysia also grows accordingly. Malaysia’s 2006 GDP is approximately 151 Billion USD (Department of Statistics 2007). Remondo et al. (2005) mentioned that, apart from climatic factors that increase landslides, socioeconomic activities also play an important role. Likewise, in the case of Malaysia, the number of landslides that affect humans is related to economic and construction sector growth. Figure 4 shows the relationship between landslides and the construction sector growth. It should be noted that the peak landslide events occurred during the early 1990s until 1997, after which the construction sector growths began to decline.

Figure 3: Geology of Sabah

The rate of development was the highest in the capital, Kuala Lumpur and its vicinity. Although more than 70% of Kuala Lumpur is relatively flat, however, at the periphery of the city, the terrain is quite hilly. Mass hill-site development became more viable due to the scarcity of land and rising land cost. The chart presented in Figure 5 shows the percentage of landslides according to the states in Malaysia. Kuala Lumpur (also known as the Federal Territory) has the smallest land area and yet experienced the most landslides — contributing more than one-fifth of the total number of landslides from 1975 to 2004. Figure 6 shows part of Kuala Lumpur and its periphery in 1990 and in 2007. It can be seen that the development has started to encroach into the hilly areas (construction areas are cleared of vegetation) in 1990 and development was quite well established by 2007.
Figure 4: Relationship between construction sector growth and landslides

Figure 5: Landslide distribution according to states from 1975 to 2004
The main reason for the increase in landslides was due to indiscriminate development and lack of control on the planning, design, construction and maintenance of the slopes. However, after 1993 when the first recorded major landslide occurred, the public became more aware of the danger due to landslides. This was followed by a series of landslides until...
the end of 2003. The landmark landslides and their ‘contribution’ to the slope management development are presented in the subsequent sections.

**Landmark Landslides and Their Impact**

Landslide records in Malaysia until recently were based on newspaper reports dated as far back as 1966. As mentioned previously, the first major landslide occurred in 1993, when a condominium block toppled over due to the undermining of the condominium foundation as a result of a landslide. There were several landslides that can be categorized as landmark landslides in Malaysia. The reasons why these landslides are considered ‘landmark’ because of a combination of the following factors:

- The landslide created awareness among the public and the professionals alike
- It is often quoted by landslide engineers and scientists in the country
- A committee has to be formed to investigate the landslide
- The government has to take some actions (or seems to be taking some actions)
- High number of fatalities and/or huge economic losses

A list of landmark landslides that satisfied most of the criteria mentioned above is presented in Table 1. There are other landslides that killed more people but yet were not included in the list because their impact is less significant.

<table>
<thead>
<tr>
<th>Landslide Location</th>
<th>Date of occurrence</th>
<th>Fatality (No.)</th>
<th>Injury (No.)</th>
<th>No. of people affected</th>
<th>Road closure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highland Towers</td>
<td>11 Dec 1993</td>
<td>48</td>
<td>-</td>
<td>&gt; 100</td>
<td>-</td>
</tr>
<tr>
<td>Genting Sempah</td>
<td>30 Jun 1995</td>
<td>22</td>
<td>20</td>
<td>thousands</td>
<td>Yes</td>
</tr>
<tr>
<td>Taman Hillview</td>
<td>20 Nov 2002</td>
<td>8</td>
<td>-</td>
<td>&gt; 10</td>
<td>-</td>
</tr>
<tr>
<td>Bukit Lanjan</td>
<td>26 Oct 2003</td>
<td>-</td>
<td>-</td>
<td>&gt; millions</td>
<td>Yes</td>
</tr>
</tbody>
</table>

**Highland Towers, 1993**

The first widely quoted landslide for Malaysia, and has since become a landmark, occurred when one of the three condominium blocks toppled over on December 11, 1993. As a result of the incident, 48 people were killed and the residents of the other two apartment blocks were asked to evacuate when the government declared the still standing blocks to be unsafe. The Ampang Jaya Municipal Council (MPAJ) assembled experts in the country to carry out an investigation of the failure. The geotechnical investigation was headed by the Public Works Department of Malaysia (JKR). This is the first time that a technical team was assembled to investigate a landslide in Malaysia. A report, entitled “Report on the Inquiry Committee in the Collapse of Block 1 and the Stability of Blocks 2 and 3 Highland Towers Condominium”, was published by MPAJ (1994). The report concluded that the most probable cause of the collapse of the condominium block was due to buckling and shearing of the rail piles foundation induced by the movement of the soil. There was a long litigation that was only recently settled. No concrete actions were taken by the government to mitigate recurrence of similar incident immediately after the event, however, it created awareness among the public especially the communities that live in hilly areas. Figure 7 shows the condominium block toppling over during its foundation was undermined by the landslide.
Debris Flow at Genting Sempah, 1995
A debris flow struck a semi-private road that led to a popular highland resorts in a place called Genting Highlands. The debris flow struck several vehicles that were pushed into a ravine, as a result of which 22 people were killed and 20 were injured. Investigation by Geology Department (now known as Geology and Minerals Department) of Malaysia revealed that the debris flow was due to successive failures of natural dams uphill to location where the debris flow hit the road. The dam failures were triggered by intense and prolong rainfall. The road has to be closed and a new road was constructed in a new alignment. As a result of the incident, the company that own the highland resorts appointed consultants to carry out inspections of the slopes along the length of the road leading to the highland resorts. Regular maintenance was carried out on the slopes along the road. No concrete actions were taken by the government to mitigate similar incident from happening. However, the incident created more awareness on the peril of debris flows and their possibility of occurrence even when the slopes are not disturbed.

Taman Hillview, 2002
The location of this landslide is approximately 300 m away from the 1993 Highland Towers. Due to a landslide flow, a house located at the foothill with 13 people were trapped in the wreckage. Out of the 13 people, 8 of the occupants of the house perished. Prior to this incident, the National Disaster Management Committee has instructed JKR to head a Working Committee on Landslide Study and JKR formed a sub-committee for forensic investigation of the landslide in August 2002. The landslide was the first time that the sub-committee on forensic investigation met which comprised of various agencies that has some association with landslide such as the Geology and Mineral, Meteorology, Drainage and Irrigation, Survey Departments and other agencies. Each department was given a specific task to perform, for example, the Geology and Mineral Department would provide geological information and perform detailed geological investigation of the area concerned, the Survey Department would look at the topography and the history of development of the area based on land survey, aerial and satellite images.
After the investigation, the sub-committee recommended strengthening of the slope be carried out and improvement to the drainage system on the slope. Both recommendations were accepted by the government but were executed a few years after the incident - the work has just been completed. The slope is also being monitored for any movement.

**Bukit Lanjan, 2003**

By far, Bukit Lanjan rock fall is the most significant landslides, because the government began to take notice of the effects that similar landslides have on the economic and social conditions on the nation especially those affecting major cities. Bukit Lanjan or Lanjan Hill is located along a very busy expressway leading to Kuala Lumpur from the north. In October 2003, the extensive rock slope failure, which estimated to be 35,000 m$^3$ in volume cut off the expressway completely. The expressway was closed for more than six months with losses to the expressway concessionaire company amounting to more than several millions USD per month. The repair works cost more than ten million USD. The immediate effect of this landslide was the government began to realize the need to have at least an agency that will look into the management of slopes in Malaysia. As a result, Slope Engineering Branch (CKC) was established in the JKR in February 2004, four months after the incident. In the same year, the Cabinet instructed the Slope Engineering Branch to carry out a National Slope Master Plan (NSMP) Study to manage all slopes in Malaysia. The study was started in March 2006 and expected to be completed in March 2008.

**PUBLIC AWARENESS AND SLOPE MANAGEMENT SINCE 1993**

**Public Awareness**

Since 1993 the public became more aware of landslides and the destruction in the wake of these landslides. Some resident associations that live near or in hilly areas have established committee that deal specifically with slope and landslide issues and they were very vocal with their demand. The residents that live in the hill-site were generally richer and more educated than the average population. They would scrutinize development in their areas and their neighborhood that may affect them directly or indirectly. Some community groups can become quite vocal at times where pickets and demonstrations were held near the area that was being developed.

The media also plays an important role in creating awareness about landslides. Sometimes the issues were highlighted because they were related to environmental issues, i.e. indiscriminate cutting of trees and erosion on the hillsides. After 1993, even small landslides that do not cause major disruption to the public would still get into the news.

When the Slope Engineering Branch (CKC) was set up, one of the units in the Branch was Training and Public Awareness Unit that deals with matters pertaining to public information and public broadcast. The approach that will be taken by CKC to provide awareness and education to the general public is through the media, by conducting exhibitions and education in schools for children. The Unit has yet to begin its program fully, therefore the success will not be known until the program is fully developed.

**Slope Management in the Public Sector**

The first document on guidelines in hilly areas development was produced by Urban and Rural Planning Department in 1997. The guidelines address the issues of planning and development in highlands, on slopes, natural waterways, and water catchments areas.
In June 2002, Geology and Minerals Department of Malaysia produced guidelines on hill-site development. The guidelines considered the angle of the natural slopes and geology of the area. The areas were then classified into 4 categories which were termed as Class I, II, III and IV. Class I is the least severe in terms of terrain grading whereby slope angles are less than 15°. Class IV represents the highest risk, where absolutely no development will be allowed in this classified area.

Also in June 2002, the National Disaster Management Committee in the Prime Minister’s Department directed JKR to form a Working Group on Landslide Study with the objectives of identifying areas with high landslide risks to come up with mitigative measures. The Working Group was officially inaugurated on August 14, 2002 and subsequently 4 sub-committees were formed, namely:

- Sub-committee for Forensic Investigation of Landslides
- Sub-committee for Disaster Management during Landslides
- Sub-committee for Co-ordination and Information Sharing
- Sub-committee for Research Coordination.

The first task given to the committee was to investigate the Taman Hillview landslide that killed 8 people that occurred at the end of 2002. The overall mandate given to JKR during this time was not very successfully implemented because the Slope Engineering Unit was then placed under the Road Maintenance Division where it has to compete for attention and resources within the division.

The CKC was established as a branch within the JKR in February 2004 with the aims of managing and monitoring of slopes throughout the country. CKC has 6 units that deal with slope matters. They are the Slope Safety Unit that would coordinate and control the budget for the slope repair works; the Slope Management Unit that would collect spatial and non-spatial data and produce hazard maps for slopes; the IT and Documentation Unit whose job is to archive and disseminate slope data and information through the website and archiving; Research and Development Unit that has functions including research, initiating cooperation with universities (local and abroad) and conducting National Slope Master Plan study; the Forensic Unit is responsible for landslide investigation and prepares standards and guidelines for slope design, the Quality, Training and Public Awareness Unit is responsible for training personnel in JKR and creates awareness amongst the public. The annual budget of CKC is about 14 million USD, most of the budget was spent on slope maintenance and repair of Federal roads.

Environment Department of Malaysia also produced development guidelines for highlands in December 2005. Highlands is defined as areas which lie 300 m above the mean sea level. The guidelines were prepared in collaboration with Geology and Mineral Department of Malaysia.

At the state level, the Selangor state, one of the states that has high landslide incidents, introduced a standard procedure of processing the application of development in environmentally sensitive areas and the highlands. The standard procedure for development in hill-site was introduced in October 2006.
Slope Management in Private Sector and Professional Institutions

The institution of Engineers, Malaysia (IEM) under its own initiative formed a task force in 1999 to formulate the policies and procedures for mitigating the risk of landslides in hilly terrain development. IEM (2000) produced a report, entitled “The Policies and Procedures for Mitigating the Risk of Landslide on Hill-site Development”, with the aims to providing a uniform, consistent, and effective policies and procedures for the consideration and implementation by the Government of Malaysia. In the position paper, the slopes for the hill-site development to be classified into 3 classes, viz. low, medium and high risk. The category of risk is based on the geometry of the slopes such as height and angle of the slope. The category of risk also includes the type of building that is going to be developed on the hill-site. The position paper also recommended that qualified and experienced consultants should audit engineers design for major development in high risk area; appointment of resident professional engineer to supervise construction; and developers, contractors and supervisors to be made accountable to the authorities for construction safety. The position paper also recommended that a special agency be formed to assist Local Governments with respect to hill-site development. However, the recommendation by IEM was not immediately accepted and acted on by the Government.

On the other hand, in the construction industry, no self regulations on industrial practices were forth coming after 1993 but increasing use of more recent techniques such as soil nailing to prop up steeper slopes became more widespread. Some companies that have extensive interest in highland areas would appoint engineers to inspect high hazard slopes and at the same time regular maintenance would be carried out on these slopes. They include expressway concessionaire companies and some highland hotels and resorts.

National Slope Master Plan

The Malaysian Government has instructed JKR to carry out the NSMP study in May 2004. The goal of National Slope Master Plan Study is to provide detailed elements of a comprehensive and effective national policy, strategy and action plan for reducing losses from landslides on slopes nationwide including activities at the national, state and local levels, in both the public and private sectors. The NSMP study was officially inaugurated in March 2006, however, the groundwork for NSMP study began a year earlier. The study is expected to be completed by March 2008 with a total estimated cost of approximately USD 1.8 million. The NSMP Term of Reference was partly based on USGS Circular 1244 (2003) and works by Committee on the Review of the National Landslide Hazards Mitigation Strategy Board of the United States National Research Council (2004).

A consortium of 6 consultants was appointed to carry out the NSMP study. The NSMP consisted of 10 key objectives which were translated into 10 components of the study. The components of the NSMP and the summary of their objectives are as follows:

1. Policies and institutional framework – to improve policies and institutional framework
2. Hazard mapping and assessments – to develop a plan for mapping and assessing landslide hazard and also develop standards and guidelines for landslide hazard mapping
3. Early warning and real-time monitoring system – to develop a national landslide hazard monitoring, prediction and early warning system
4. Loss assessment – to assess the current data on landslide losses and develop national plan for compilation, maintenance and evaluation of data from landslide
5. Information collection, interpretation, dissemination, and archiving – to evaluate the state-of-the-art technologies and methodologies for dissemination and archiving of technical information
6. Training – to develop training programs for personnel involved in landslides
7. Public awareness and education – to evaluate and develop education program related to predictive understanding of landslide
8. Loss reduction measures – to evaluate and develop effective planning, design, construction and maintenance with the view for landslide hazard reduction
9. Emergency preparedness, response and recovery – to develop a national plan for a coordinated landslide rapid response capability
10. Research and development – to develop a predictive understanding of landslide processes, threshold and triggering mechanisms

From the components and the objectives, it can be seen that the study provide a very comprehensive coverage of issues pertaining to slope management.

The NSMP is to be implemented in 3 phases: the first phase is called the short term which would cover the first 5 years; the second phase is known as the medium term i.e. the period of implementation between 5 and 10 years; and final phase which is known as the long term which is the period of implementation of 10 to 15 years and beyond. The success of the NSMP depends on the political will, budget allocation, collaboration between stakeholders, and the relevancy of the suggested plans.

SUMMARY AND CONCLUSIONS
The history and progress of slope management in Malaysia since 1993 has been presented. Some landmark landslides and their impact on slope management were offered. The awareness on landslides among the public began in 1993 when a devastating landslide toppled a condominium block that killed 48 people. The first forensic investigation was initiated as a result of this event. No immediate action was forth coming out of this incident either from the government or the private sector. After the debris flow in 1995, the highland resort that is directly affected by the event employed engineers to carry out periodic inspections on slopes along the road leading to the resort. Regular maintenance was also done on the slopes. Expressway concessionaire companies also have special engineering teams that would carry out inspections of slopes along their expressways.

After successive major landslides since 1993, the JKR was given the task to establish a dedicated branch that would solely focus its attention on slope related issues. This action came about after a rock slope failure that cut off the expressway for more than 6 months and caused daily traffic congestion in the capital city, Kuala Lumpur, and huge losses to the expressway concessionaire. From the landslide chronology and events mentioned, it is clear that, as in Hong Kong, concrete actions by the government, perhaps more than the case in landslide, needs several triggers before appropriate action are put in place. The setting up of CKC culminated in the NSMP study that would address most of the issues on slopes faced by the nation. The success of the NSMP is yet uncertain but would depend upon the government, CKC, the engineers and scientists, and not least the public.
REFERENCES

ACKNOWLEDGEMENTS
The authors would like to acknowledge JKR personnel who have assisted in the preparation of this paper.
LANDSLIDE RISK MANAGEMENT IN HONG KONG

R. K. S. Chan and S. H. Mak
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Abstract: Hong Kong’s steep hilly terrain coupled with intense infrastructure and building developments and heavy rainfall in the summer have resulted in acute slope safety problems in the past. The Geotechnical Engineering Office of the Civil Engineering and Development Department has developed a comprehensive Slope Safety Management System, which has been continuously enhanced and updated in the light of experience gained in the past 30 years. The System comprises seven key result areas by improving slope safety standards and technology, ensuring the safety of new slopes, improving the safety of existing slopes, enhancing slope appearance, promoting proper maintenance of slopes and providing public education, publicity, and public warnings as well as information services on slope safety. This paper describes the background, rationale and evolution of each key result area, key technical and administrative challenges, experience gained and effectiveness and future strategic directions to meet Hong Kong’s needs for the highest standards of slope safety.

INTRODUCTION
Hong Kong has a land area of about 1,100 km², less than 40% of which is flat land (Figure 1). With the steady increase in population from 2 million in 1950 to 6.9 million in 2007 (Figure 2) and rapid economic growth, there has been a continual huge demand for land for housing and infrastructure developments, a substantial portion of which are located on or near to steep hillsides (Figure 3). The result is the formation of some 57,000 sizeable man-made slopes and retaining walls (Figure 4). The high seasonal rainfall of 2,300 mm a year mostly falls in the summer and often triggers landslides leading to loss of life and damage to properties. The challenges that Hong Kong has been facing with respect to slope safety are unique.

Figure 1: Development of a city on hilly terrain (Hong Kong)
Figure 2: Steady increase in the Hong Kong population since 1950

Figure 3: Residential development on a 50-m high slope in Hong Kong

Figure 4: High concentration of man-made slopes in Hong Kong
DAWN OF THE HONG KONG GEOTECHNICAL ERA

In the late 1960’s and 1970’s, there had been repeated landslide disasters, which claimed many lives. During the torrential rainstorms that hit Hong Kong on 12 June 1966, disastrous landslides struck many parts of Hong Kong including densely populated housing estates and transport links (Figure 5).

Figure 5: A tangled heap of vehicles in a busy street after the torrential rainstorm that hit Hong Kong on 12 June 1966

64 people died and a further 48 went missing with more than 6,000 people directly affected by the disaster. The 6.18 disaster, which took place on 18 June 1972, made it the darkest day in the history of landslide disasters in Hong Kong. 653 mm of rain was recorded by the Hong Kong Observatory between 16 and 18 June. Tragedies struck one after another. The greatest damage was done in a temporary resettlement area at Sau Mau Ping in eastern Kowloon (Figure 6) and in the high class residential area at Po Shan Road on Hong Kong island (Figure 7) killing 71 and 67 people respectively (Government of Hong Kong 1972). The government started to place some importance on slope safety and allocated some dedicated resources to tackle the geotechnical problems arising from slopes. A few “geotechnical engineers” with civil engineering background were engaged in geotechnical control of private building works and overall review of the landslide problems.

Figure 6: Landslide at Sau Mau Ping in 1972 (71 fatalities)
As the fear of landslides seemed to have slightly relieved, another disastrous landslide occurred in Sau Mau Ping (Figure 8) again on 25 August 1976 taking away the lives of 18 residents in a public housing estate. Government reacted positively with the appointment of an independent panel of local and international geotechnical experts to investigate the Sau Mau Ping disaster and to make recommendations to tackle the slope safety problems in Hong Kong. One of the key recommendations of the Panel was the establishment of a government geotechnical control body, the former Geotechnical Control Office (now renamed the Geotechnical Engineering Office (GEO)), in 1977 (Government of Hong Kong 1977). With the concerted efforts by GEO and related government departments/bureaux and the community, a comprehensive Slope Safety System has been set up to reduce landslide risk to as low as reasonably practicable. Key elements of the Hong Kong Slope Safety System are discussed hereunder in terms of the background/rationale and evolution over the past 30 years, the technical challenges, experience gained, effectiveness and future strategic directions.

KEY COMPONENTS OF THE SLOPE SAFETY SYSTEM
The Hong Kong Slope Safety System has seven key components to deliver the overall policy objective to meet Hong Kong’s needs for the highest standards of slope safety. The seven Key Results Areas (KRAs) are:

- KRA 1. Improve slope safety standards, technology, and administrative and regulatory frameworks.
KRA 2. Ensure safety standards of new slopes.
KRA 3. Rectify substandard government slopes.
KRA 5. Ensure that owners take responsibility for slope safety.
KRA 6. Promote public awareness and response in slope safety through public education, publicity, information services and public warnings.
KRA 7. Enhance the appearance and aesthetics of engineered slopes.

Experience shows that our dedicated efforts in the above seven areas in the past 30 years have brought about substantial improvement in the safety of slopes in Hong Kong and are discussed under each KRAs below.

**KRA 1: Improve Slope Safety Standards, Technology, and Administrative and Regulatory Frameworks**

**Geotechnical Standards**
The Sau Mau Ping landslides in 1972 and 1976 were principally due to heavy rainstorms and the presence of loose fill slopes formed by end tipping in accordance with engineering practice in the 1960s. Whilst rainfall is natural occurrence and could not be controlled, good slope safety standards and technology are indispensable to the creation of a safer built environment. Before the 1970s, slopes were mainly formed by empirical means based on rules of thumb, such as 55° for soil cut slopes and 35° for fill embankments. There was little geotechnical input, except for critical facilities, such as dams (Chan et al. 2007b). Prompted by the disastrous landslides in the 1970s, engineers recognized the need to design slopes to an acceptable standard yet to be developed. A practical approach is to adapt the practice in other countries for the investigation and design of dams/slopes to Hong Kong (Beattie & Chau 1976). With experience gained, the first edition of the Geotechnical Manual for Slopes (GCO 1979) was published in November 1979 as a draft, to promote discussions among and comments from those concerned with construction in Hong Kong. The second edition of the Manual (GCO 1984) was then published in May 1984 to introduce some improvements to the first edition and to update the geotechnical engineering standards and practice to take account of improvements in practice since 1979. This set the scene for a consistent approach and good standards of practice that should be adopted for the design, construction and maintenance of slopes and site formation works in Hong Kong. The publication of the Manual marked an important step to enable the geotechnical profession involved in slope formation and rectification to work towards common standards commensurate with the expectation of the Hong Kong community on slope safety. For example, fill would have to be compacted in horizontal layers instead of end tipping to form new fill slopes; existing loose fill slopes would be rectified by re-compacting the top 3m of the fill (Figure 9) (Bowler & Phillipson 1982).
Since 1984, updating of various topics in the Manual has been done by publication of Geoguides, e.g. Geoguide 1 – Guide to Retaining Wall Design (GEO 1993). Up to the present, a total of 6 Geoguides have been produced. In addition, technical guidelines, state-of-the-art reviews and technological advancements are regularly promulgated via GEO Publications, Geospecs, Geological Publications, Geological Memoirs, GEO Reports and GEO Technical Guidance Notes. Over 150 of these documents which are now available for free download from the CEDD Website (http://www.cedd.gov.hk) form a rich pool of geotechnical reference materials for the geotechnical profession both locally and overseas. The net result is a vibrant local geotechnical community with good up-to-date knowledge of the requirements on the geotechnical works in Hong Kong.

**Technical Developments**

In the past 30 years, there have been significant improvements in slope engineering practice through technological advances, continuous improvement initiatives and lessons learnt from landslides. The latest advancements are in the development of systematic landslide investigation, enhancement of robustness of engineered slopes, improved understanding of the behaviour of soil-nailed slopes, use of Quantitative Risk Assessment, mitigation of natural terrain landslide hazards and application of novel technologies (Chan et al. 2007b).

Since 1997, systematic investigation of landslides has provided valuable information on the actual performance of slopes, since landslides are regarded as full-scale destructive tests of slopes in this regard. Outcomes of landslide investigation have contributed to enhancing the Slope Safety System through identifying the causes of the failure and the characteristics and the types of slopes that deserve special attention, making slopes more robust and identifying areas for improvement in design and construction practice and pointing the direction of technical developments (Ho & Pappin 2007).

**Administrative and Statutory Geotechnical Control Frameworks**

To ensure the safety of developments, the geotechnical control framework has been continually enhanced. For public developments, the mandates for geotechnical control are...
derived from administrative circulars issued by the Development Bureau and are available on the Bureau Website (http://www.devb.gov.hk). Works departments/offices have to comply with the stipulated requirements in terms of geotechnical standards and procedures. Cooperation by the works departments/offices is needed with the support from the Bureau.

For private developments, the control framework is the statutory powers of the Buildings Ordinance and related Regulations (Government of Hong Kong 2007). Back in 1977 at the time of formation of the GEO, there were very limited laws and regulations relating to the geotechnical aspects of private developments. Noting that the Po Shan landslide in 1972 had been mainly triggered by inadequate support to excavation in a construction site, it was imperative to enhance the statutory control on the geotechnical aspects of private developments. Various laws and regulations have been passed since 1980’s to stipulate the statutory requirements in terms of the investigation, design and construction of private geotechnical works, e.g. site investigation, site formation, excavation and lateral supports and foundations. For areas of special concerns, Scheduled Areas have been delineated with special statutory geotechnical control to abate the concern (Chan & Chan 1998).

In the Mid-levels where the disastrous Po Shan landslide occurred in 1972, there was serious concern on the cumulative effects of further development in the area on the regional stability of the hillside. A statutory ban on building developments was imposed in the Mid-levels area in 1979 to enable a comprehensive geotechnical study of the area. Based on the results of the Mid-levels Study (GCO 1982), the Buildings Ordinance and related Regulations were amended to include special geotechnical control in the Mid-levels area prior to lifting of the ban in 1982. Such statutory geotechnical control includes requirements on bulk excavation limits, design of buildings to resist landslide debris, groundwater drainage works beyond development site boundaries and performance reviews. The restriction of bulk excavation for developments (Figure 10) in the Mid-levels area is necessary to minimize cumulative adverse effects of bulk excavation on the stability of the hillside generally. Experience in the past 25 years has shown that these stringent statutory requirements are appropriate to ensure safe developments in the geologically complex and steep hilly area of the Mid-levels.

Figure 10: Restriction of bulk excavation for developments in Mid-levels area, Hong Kong
In the northwest New Territories and Ma On Shan, the underlying bedrock is marble with variable extent of dissolution or cavities (Figure 11). GEO carried out a series of studies in the late 1980's to examine the scale of foundation problems posed by cavernous marble, to define the nature of risk to foundations, and to develop safe practice (Chan 1996). Based on the results of the studies, the practice of extracting groundwater from deep wells has been discontinued and special geotechnical control for foundation design and construction in marble has been implemented, including performance review of foundations prior to issue of occupation permit. Guidelines on the investigation, design and construction of deep foundations on marble are also provided to practitioners, e.g. Chan (1994), GEO (2005a) and GEO (2006).

Throughout these years, bills to enhance statutory geotechnical control have gained support from the industry as well as legislators. The latest initiative approved by the Legislative Council in 2004 was the setting up of a list of Registered Geotechnical Engineers (RGE) under the Buildings Ordinance. It is now a statutory requirement that only qualified geotechnical engineers are responsible for the geotechnical aspects of private building works, hence ensuring quality of geotechnical input, PNAP 294 (BD 2006).

As compared with the administrative control system for public developments, the statutory geotechnical control framework for private developments is more effective but any improvement to the legal framework normally takes a few years from formulation of the initiative to enactment of the legislation.

KRA 2: Ensure Safety Standards of New Slopes

Introduction
In the course of building and infrastructure developments in Hong Kong, many new slopes are formed every year amounting to a total of some 18,000 slopes constructed after the establishment of the GEO in 1977. The design and construction of new slopes to acceptable standards is one of the key elements in ensuring their long-term stability and creating a safe and sustainable built environment for the Hong Kong community. With the legal and administrative powers available to the GEO, a comprehensive Geotechnical Control System has been set up to audit the adequacy of the design and construction of all geotechnical works including slope works, site formation, earth retaining structures, deep excavations and tunnels.
by the private sector, public authorities and government departments. The audits are being carried out by geotechnical engineers in the GEO as contrast to the use of independent checkers from the private sector elsewhere in other countries. In 2006, there has been a debate in the industry on whether building control work (including geotechnical control) should be delegated to “independent” private checkers. A number of issues have been identified as follows:

(a) difficult to ensure quality of checking by independent private checkers;
(b) independent checkers do not have good district knowledge and experience gained on problems encountered in various types of developments;
(c) independent checkers do not have an overall view and hence their inter-relationship of developments in the district; this may have serious public safety implications, e.g. too intense developments in a steeply sloping area could give rise to overall stability problem, cumulative effects of excavations and damming on groundwater;
(d) concern on professional liability and high professional indemnity insurance (PII) could result in very small monopolistic group of independent checkers;
(e) impartiality and conflict of interest of independent checkers;
(f) inconsistency in checking (non-level playing field); and
(g) lack of public confidence in independent checkers.

It was concluded that the above issues could compromise public safety and there is no quick solution. Further work on the feasibility of self-certification by independent private checkers has been held in abeyance.

**Geotechnical Control at Planning Stage**

To facilitate developments, the GEO has been providing advice to relevant government departments as early as the planning stage where site usages could be easily interchanged to avoid geotechnically difficult areas. For example, residential development sites could be located in areas less prone to geotechnical problems when the zoning plans are being formulated (Figure 12). At the time of preparing the land-title documents, the GEO makes recommendations on special geotechnical conditions to forewarn the private/public developers of the range of geotechnical problems likely to be encountered, to enable the developers to reserve adequate resources to address these issues. During the private building conceptual planning stage, developers are required to engage RGEs to evaluate the geotechnical feasibility of the proposed development so that suitable changes could be made when the building layout is still on the drawing board. All these measures play an important role to help developers to avoid embarking on geotechnical difficult schemes, which could lead to unnecessary geotechnical risk during construction and in the long term (Chan et al. 2007b).
Figure 12: (a) Large scale debris flow in 1990; (b) Following the 1990 debris flow, area rezoned from residential development to a golf driving range

Geotechnical Control at Design and Construction Stages
In addition to ensuring public safety, audit of geotechnical designs in developments has the benefit of maintaining consistency in design practice to the required safety standards. Another advantage is identification of general deficiencies in geotechnical practitioners. In collaboration with local universities and institutions, suitable courses/seminars have been organized to help practitioners in continuous professional development. Practice Notes and Technical Guidance Notes are regularly issued to provide administrative and technical guidance.

No matter how good a design is, the most important outcome is what is actually constructed on site. For geotechnical works, the geological and hydro-geological conditions could not be fully revealed from site investigation alone. Actual conditions exposed during construction provide pertinent information for the designers to decide whether geotechnical assumptions made are valid or design amendments have to be made. The Hong Kong practice is to require supervision of the works by technically competent persons and regular inspections by the designer (BD 2005). GEO district geotechnical engineers also carry out periodic site audits. For geotechnically sensitive sites, performance review may be stipulated at the approval stage to require the designer to demonstrate that the geotechnical works have been adequately inspected and monitored in the course of construction and that the geotechnical design assumptions upon which the geotechnical works have been based are valid.

For active construction private sites, statutory powers are available to issue Cease Work Order under situations of imminent danger and Government can step in to carry out necessary emergency works to ensure public safety. Similarly, for public developments, the Project Department will take emergency actions with advice from GEO if needed.

It can be seen that the Hong Kong Geotechnical Control System has been well established over the past 30 years and experience shows that the success rate in preventing major landslides in slopes audited as conforming to the required safety standards is well in excess of 99.8%.
KRA 3: Rectify Substandard Government Slopes

Introduction

Prior to the establishment of the GEO in 1977, there was very limited geotechnical control of slope formation both in the private and public sectors. The stability of many old slopes is therefore in doubt. With increased attention to the slope safety problems in the 1970s, the first major task was to identify the size of the problem by compiling a catalogue of sizeable man-made slopes in the urban area and along motorable roads in the New Territories, i.e. failure of these slopes could have serious consequences on lives and properties. In the late 1970s, the Catalogue contained pertinent information on some 10,000 slopes, which may affect developments and major roads. A major re-cataloguing exercise in 1995 registered some 54,000 slopes (about 36,000 pre-1977 slopes) in the whole territory of Hong Kong (Mak et al. 2001). As at 2007, the number of registered slopes is some 57,000. Since the early 1980s, the GEO has taken a proactive role in rectifying substandard government slopes under the Landslip Preventive Measures (LPM) Programme (Figures 13 & 14). Expenditure on investigation and slope works under the LPM Programme has steadily increased in the past 15 years with some HK$1 billion budgeted for 2007.

Figure 13: Forecast and actual number of slopes upgraded under the LPM Programme

Figure 14: Forecast and actual expenditure under the LPM Programme
With a large number of slopes, it would not be possible to investigate and upgrade all of them within a short time. A risk-based priority system based on consequence of failure and stability conditions was developed to enable slopes with higher risk to be upgraded first, thereby achieving maximum risk reduction in the shortest possible time and progressively reducing the risk from slopes which affect the community directly. Urgent repair works would be implemented for slopes which exhibit signs of distress, so that the temporary stability of the slopes in question is restored pending upgrading works in due course. Regular routine maintenance would be carried out to reduce the chance of deterioration of the slopes. The LPM strategy has proved to be effective in reducing landslide risk to the community and demonstrated good commitment from Government to combat landslides (Tang et al. 2007a).

**Retrofitting Man-Made Slopes**

Initial attention was focused on loose fill slopes, which could have disastrous consequence, as reflected in the fill slope failures in Sau Mau Ping in 1972 and 1976. The conventional method is to re-compact the top 3m of loose fill with drainage provided at interface of the loose fill and the re-compacted fill (Figure 9). This has proved to be successful in reducing the risk from loose fill slopes (Law 2001). With the increasing public demand on improvement to the built environment, trees have to be preserved as far as possible in developments including slope works. Re-compaction of fill will normally require removal of existing trees, which could lead to serious public outcry. To meet public expectation on preservation of trees and provision of a green environment where possible, the use of soil nails to stabilize loose fill slopes has been developed in early 2000’s through a joint effort between the GEO and the Geotechnical Division of the Hong Kong Institution of Engineers. Guidelines for the use of soil nails in loose fill have been promulgated through the Final Report on Soil Nails in Loose Fill Slopes (HKIE 2003). The soil nailed fill slopes could then be greened with existing trees preserved (Figure 15).

![Figure 15: Soil-nailed fill slope with existing trees preserved](image_url)
Cut slopes warrant similar attention in comparison with fill slopes since the consequence of failure could also be very serious, as evidenced by the Po Shan Road failure in 1972. Based on the Slope Catalogue, there are over 44,000 cut slopes and retaining walls, 39% of which could affect developments and busy roads. In the 1980s, slopes were normally formed or upgraded through cutting to gentler angles resulting in large cut slopes without any structural support. In some cases, steep slopes were retained by prestressed ground anchors (Figure 16). However, the need for recurrent maintenance and monitoring of anchor loads throughout their design life has forced designers and slope owners to adopt permanent anchors only in exceptional circumstances. In fact, no permanent anchors have been used in slopes in Hong Kong since the late 1980s. Other types of stabilization measures, such as stabilizing piles and retaining walls with backfill have been adopted where site conditions are suitable. There was a need to develop a slope stabilizing method, which is robust, cost-effective and easy to design and construct. Watkins & Powell (1992) described the use of soil nails in landslip preventive works in Hong Kong. Since the mid-1990s, soil nails have been widely used to stabilize new and existing slopes with over 80,000 soil nails installed each year (Figure 17). Review of the soil-nailed slopes in Hong Kong (Ng et al. 2007) indicated that from 1977 to 2006, the average annual failure rate of minor landslides on soil-nailed slopes is about 0.078%. No major failure (volume ≥ 50 m$^3$) on soil-nailed slopes has been reported in Hong Kong since the introduction of soil nails for slope upgrading in the late 1980s. The above review has demonstrated high robustness of the soil nailing as compared to unsupported cut (Figure 18). The design practice and technical developments of soil nailing in Hong Kong are given in Pun & Shiu (2007).
Mitigating Natural Terrain Landslide Risk

Over 60% of Hong Kong is covered by natural terrain which has been subject to minimal human disturbance from the public. Although the main focus of the Hong Kong Slope Safety System has been on landslide risk from man-made slopes, the risk from natural hillsides is increasing due to new developments encroaching on steep natural hillsides and strong resistance to reclamation in recent years. The GEO has been carrying out systematic research and studies on natural terrain hazards in Hong Kong, with the compilation of the natural terrain landslide inventory based on aerial photograph interpretations (Figure 19). Study and mitigation of natural terrain hazards affecting existing developments are carried out following the ‘react-to-known-hazard’ approach. Undue cumulative increase in the risk to new developments is controlled through avoidance of development in hazardous areas as far as possible, and study and mitigation of hazards as part of new developments where required.

With a vast extent of natural terrain covering an area of over 660 km$^2$, there is a need to build up an inventory of the natural hillsides which could have public safety concerns. Based on examination of both high-flying and low-flying aerial photographs covering the whole territory of Hong Kong, some 2,700 catchments of the natural hillside have been identified with a known history of failures affecting existing developments and major roads. It is anticipated that the number of vulnerable natural terrain catchments close to existing developments will increase at an average rate of about 10 per year, due to the occurrence of landslides from time to time.

Figure 19: Example of the natural terrain landslide inventory
In mitigating natural terrain landslide risk, the greatest challenge is to resolve the environmental constraints without compromising safety. Environmental issues could include preservation of trees and rare species of plants, insects and animals. Even if these issues could be satisfactorily dealt with, it could take a long leading time for actual works to start.

The two main types of natural terrain landslides are debris flow and open hillside failures. For debris flow, the common mitigation measures include the erection of debris-resisting barriers, e.g. check dams (Figure 20), to protect developments on the downstream side of the debris through containment. The key issue is to assess the likely debris volume to be retained. Open hillside failures could be stabilized by soil nails with re-grading of localized steep profiles or installation of debris-resisting barrier (Figure 21), but it would be difficult to clearly delineate areas which could be subject to open hillside failures; extensive soil nailing could also have adverse impact on the environment. Details of the latest development in natural terrain landslide risk management are given in Wong & Ho (2006).

![Figure 20: Check dam in Sham Tseng San Tsuen, Hong Kong](image)

![Figure 21: Debris-resisting barrier in Hong Kong](image)

**Applying Quantitative Risk Assessment (QRA) to Landslide Risk Management**

The GEO was among the first in the world to apply QRA techniques to manage landslide risk as well as to measure the performance of the Hong Kong Slope Safety System. In the past decade, QRA has been applied to global and site-specific landslide studies in Hong Kong. Global QRA was used to assess the risk due to different landslide hazards posed to the
community in order to define the relative contribution of different components to the total landslide risk. This provides a useful and valuable reference for landslide risk management, in particular, in the consideration of resources allocation and policy.

On the other hand, site-specific QRA aims at assessing landslide risk at a given site. It has been applied to problems that may not be directly amenable to conventional slope stability analysis, or where a failure is liable to result in serious consequences. It is also used to develop cost-effective mitigation strategy for landslide and boulder fall hazards from natural hillsides at specific sites.

Details of the research and engineering practice of landslide risk assessment and management in Hong Kong are presented in Tang et al. (2007b). A review of the achievements, advancement and lessons learnt in the past 30 years of landslip preventive measures is given in Tang et al. (2007a).

**KRA 4: Maintain All Government Man-Made Slopes**

**Maintenance Responsibility of Slopes**

Regular maintenance is essential for all slopes and retaining walls to avoid deterioration. As the owner of some 39,000 man-made slopes, the government has set up a comprehensive system for maintenance of government slopes. Prior to the mid-1990s, there was no readily available information on the maintenance responsibility of individual slope. One has to examine the lease/allocation conditions for the piece of land on a case by case basis, many of which necessitate specialist legal interpretation. The situation was most unsatisfactory and often resulted in delay in emergency or follow-up actions after landslides. The lack of clear responsible maintenance parties had resulted in no or very little maintenance of many slopes. To address this issue, the Lands Department, which is the authority on land management matters, embarked on a Systematic Identification of Maintenance Responsibility (SIMAR) project in the late 1990s at a cost of HK$74 million to review the lease/engineering conditions and identify the parties responsible for maintenance of all slopes registered in the Slope Catalogue. For government slopes where there is no clear demarcation of maintenance responsibility, the principle of builder-maintain and beneficiary-maintain applies. It should be pointed out that private slope responsibility is wholly determined under the provisions in the lease conditions or land-title documents and if these documents could not hold the private parties responsible, a government department will have to take up the maintenance in default. The SIMAR register has since been uploaded to the INTERNET (http://www.landsd.gov.hk/smris/index.html) and regularly updated to reflect changes in land sale/allocations or change of ownerships.

The completion of the SIMAR register in 1998 marked an important milestone in ensuring proper maintenance of slopes. The SIMAR register has greatly facilitated slope maintenance, particularly for government slopes, where maintenance departments/agents have clear knowledge of which slopes they are responsible. They could allocate appropriate resources for proper maintenance in accordance with Geoguide 5 (GEO 2003).

**Maintenance Actions**

In accordance with Geoguide 5, slopes would generally be inspected by a professionally qualified geotechnical engineer at least once every 5 years. The main purpose of the Engineer Inspection (EI) is to identify changes in conditions of a slope which will warrant follow-up actions through LPM or enhanced maintenance; emergency actions will be taken
for any immediate and obvious dangerous situation identified. EI forms a safety net in the landslide prevention loop in that slopes (irrespective of whether they have been upgraded or not) will be regularly inspected and their conditions reviewed to identify necessary follow-up maintenance or LPM actions.

To facilitate maintenance actions, GEO has published guidelines on different types of prescriptive measures to address common problems revealed during maintenance inspections. Prescriptive measures are pre-determined, experience-based and suitably conservative modules of works that are prescribed by qualified geotechnical professionals without the need for detailed investigation, laboratory testing and design analyses. Details of prescriptive measures are given in Wong et al. (1999). These measures have proved to save time and efforts for slope owners or their agents to take prompt maintenance actions.

With an aim to enhance the quality of maintenance of government slopes, the GEO carries out regular slope maintenance audit since 1996 (WB 1996). The audit generally composes of audit of the slope maintenance system in place in various maintenance departments and site audits of the state of maintenance of slopes. Recommendations for improvement are given to maintenance departments who may seek further advice if needed.

With the setting up of clear government policy on slope maintenance and the concerted efforts of maintenance departments and GEO, the annual expenditure of HK$600 million on maintenance of government slopes is well spent. Substantial improvement in the state of maintenance of government slopes has been made in helping to reduce slope deterioration.

KRA 5: Ensure that Owners Take Responsibility for Slope Safety

Introduction
In Hong Kong, there has been a building boom for several decades with large number of multi-storey buildings and housing estates constructed by private developers; a fair number of infrastructures, e.g. bridges, roads, are also built as part of housing or railway developments (Figure 22).

As many of these developments are located on hilly terrain, many man-made slopes have been formed as part of the private developments and these should be maintained by private parties. As slope maintenance is not mandatory under the provisions of the building laws in Hong Kong, GEO has made continual proactive efforts to ensure that owners take up their

Figure 22: Close proximity of steep slopes to buildings and roads
responsibility for slope safety (Massey et al. 2001).

**Rectifying Dangerous Private Slopes**

As discussed earlier, many slopes were constructed before 1977 with limited geotechnical input and some of these could be substandard. As an one-off exercise, GEO carries out safety-screening of private man-made slopes to establish prima facie evidence for serving Dangerous Hillside (DH) Orders to private owners under the Buildings Ordinance requiring them to upgrade their substandard slopes. Selection of slopes for safety-screening is based on the same risk-based priority ranking system as for LPM actions on government slopes. Initially, private slopes were systematically selected for safety-screening in accordance with their ranking scores. Some private owners received a number of DH Orders over a span of several years resulting in undue inconvenience and additional expenditure when compared to the situation of dealing with all their substandard slopes at the same time. A lot-by-lot approach has been adopted under which all slopes in a private lot will be safety-screened and DH Orders, if warranted, will be issued to the private owners. The main benefit of this approach is to let the private owners take actions on slope safety in an integrated manner.

**Voluntary vs. Mandatory Slope Maintenance**

In addition to upgrading substandard private slopes, owners should arrange for the necessary maintenance inspections and routine maintenance works in accordance with Geoguide 5 (Figure 23). There are no statutory provisions in the laws of Hong Kong to require such maintenance actions. GEO has been providing public education, publicity, information and community advisory services to encourage private owners to maintain their slopes. Details of these efforts are described in KRA 6 below.

![Figure 23: Landslides caused by lack of maintenance](image)

The alternative approach of seeking legal powers to implement mandatory maintenance of private slopes has been considered for some years. The problems associated with the policing and enforcement of mandatory slope maintenance are considerable with a chance of government maintaining a proportion of private slopes in default. The present approach of voluntary slope maintenance by private owners with assistance from government is considered to be the best approach with mandatory slope maintenance as a last resort. To gauge the need for government intervention through legislation, GEO has been assessing the state of maintenance of private slopes through site inspections as part of our slope safety work and the slope maintenance conditions are considered generally acceptable with some improvements in the past few years.
Public education on slope safety
Up to the early 1990s, many private owners wrongly regarded all slopes as belonging to government and they have no obligations to maintain their slopes. To overcome this misconception, GEO started the public education programme to instill in the minds of the public that slopes within and in the vicinity of buildings could be under private responsibility and need maintenance. The programme included talks/seminars for private flat owners, announcements of public interest in televisions and radios, and exhibitions in popular shopping centres (Figure 24). For new buildings with slopes in their vicinity, developers are required to mark clearly on their sales brochures the slopes for which future owners will have to maintain. The publication of the SIMAR register of slope maintenance responsibility also allows owners to search for the slopes they are responsible. All these measures make private owners understand that they are responsible for maintenance of their own slopes.

![Figure 24: Exhibitions on slope safety in popular shopping centres](image)

Public education is not confined to the maintenance responsibility issue. The main purpose is to promote public awareness and response in slope safety, particularly during uneventful years with relatively small number of landslides. Experience shows that the community could have short memories and tends to become complacent if no serious landslides take place for a few years. The challenge is to sustain public awareness of landslide risk through regular educational activities with special initiatives to remind the public of the importance of slope safety giving the Hong Kong hilly terrain and intense developments.

To build a good foundation for public education on slope safety, an effective approach is to include landslides as part of the school curriculum. Since the late 1990s, the subject of landslides has been incorporated in the secondary school Forms 2, 4 and 6 geography curricula. To assist teachers, GEO has produced a teaching kit on landslides, which include workbooks, worksheets, VCD and cassettes and provided free to schools for their use (Figure 25). To supplement classroom learning, GEO regularly organizes slope safety exhibitions and talks in schools (Figure 26). In the past few years, another focus is on educating children with production of cartoon books, cartoon VCD, flash cards, chesses, children umbrellas and stationeries and computer games. Students/Children could act as “Slope Safety Ambassadors” to bring slope safety messages back home to their parents and family members (Mak et al. 2007).
Regular publicity activities include Television Announcement of Public Interest (TV-API), Radio API, newspaper articles and leaflets. Professional film producers have been engaged to produce commercial type of 30-second short films using modern filming techniques with dynamic sound effects. APIs consist of 2 main categories, slope maintenance theme for broadcasting during the dry season and a theme on personal precautionary measures to be taken during heavy rainstorms for showing on TV in the summer rain. Through these TV APIs, the public and private slope owners would be reminded to take appropriate measures to enhance slope safety.

**Partnering with the Community**

Partnering with the community is the key to success in promoting public awareness on slope safety. Since 1999, GEO has set up a Community Advisory Unit (CAU) to provide community outreach to educate the public on how to tackle slope safety problems. Seminars and talks are organized for housing managers and their staff on the sources of slope information and the technical requirements for maintenance in accordance with Geoguide 5 (Figure 27). The CAU proactively approaches slope owners who receive Dangerous Hillside Orders to provide advice as necessary (Figure 28). Such service is welcome by
slopes owners particularly when they have to arrive at a general consensus to proceed with upgrading of slopes under multi-ownership. The CAU also provides ad-hoc advice on slope maintenance to help owners who voluntarily take actions to maintain their slopes.

![Figure 27: Seminar on slope maintenance for housing managers](image)

The media has been influential in raising public awareness on issues such as slope safety. GEO has been partnering with the media in promoting slope safety and personal precautionary measures. Editors and reporters are regularly updated on government slope safety work and various initiatives planned (Figure 29). Through these contacts, the media understand that landslide risk could not be reduced to zero and government has been providing adequate resources in respect of slope maintenance and landslide prevention.

In the past 2 years, GEO has extended the partnering initiative to include non-government organizations with a shared vision – “reducing natural disasters by education”. For example, the Hong Kong Red Cross (HKRC) has a mission to “deliver impartial and quality care to protect life and health and to enhance the capacity of vulnerable people to live a safe and dignified life.” By joining force with the HKRC in providing community service to residents in remote villages, volunteers from GEO and HKRC helped villagers to conduct
regular maintenance of body health and slopes alike (Figure 30). Responses from the villagers, especially the aged group, were very good (Mak et al. 2007).

![Visit of journalists to GEO](image1.jpg)

**Figure 29: Visit of journalists to GEO**

![Community activity jointly arranged by Hong Kong Red Cross and GEO](image2.jpg)

**Figure 30: Community activity jointly arranged by Hong Kong Red Cross and GEO**

**Providing Slope Information**

To facilitate maintenance, information on slopes is important. GEO has set up a computerized Slope Information System (SIS), which contains pertinent information on 57,000 man-made slopes registered in the Slope Catalogue. The SIS, which is available to the public through the Hong Kong Slope Safety Website (http://hkss.cedd.gov.hk), is one of the largest slope information database in the world with over 11 million data fields (Figure 31). The Website also contains a wealth of information and advice on slope maintenance, personal precautionary measures during heavy rainstorms, progress of government slope upgrading works, landslide investigations, natural terrain landslide hazards, etc. Through the information provided, the public could have a better understanding of the slope safety issue thereby winning their support and participation to enhance slope safety in Hong Kong (Mak et al. 2001).
To serve as a vivid reminder of the destructive consequences of landslides, GEO has collected over 200 photographs and precious information on landslides in Hong Kong for the past 100 years. A book entitled “When Hillsides Collapse – A Century of Landslides in Hong Kong” (Figure 32) (GEO 2005b) was published in 2005 with excellent feedback from the industry as well as the community. Through this book, the public could have a better understanding of the need for continued vigilance and efforts in combating landslides.

**Warnings on Landslide Danger**

Public warnings serve dual purposes of alerting the public of potential landslide danger and reminding them to take appropriate personal precautionary measures, such as “KEEP AWAY FROM SLOPES”. For slopes which are found to be dangerous or liable to become dangerous, appropriate warning signs (Figure 33) have been erected to warn the public of the potential danger. Similar warning signs are posted in squatter areas to warn those squatters (Figure 34), which have been found to be especially vulnerable to landslide risk during heavy rain (Chan et al. 2007a).
GEO maintains an extensive network of automatic raingauges to provide real-time rainfall data to the Landslip Warning System (Figure 35). Together with rainfall forecasts from the Hong Kong Observatory (HKO) and based on the spatial distribution and technical information on slopes over the territory of Hong Kong, a computer algorithm has been developed to identify instances when landslide danger is high and to issue the Landslip Warning through the media (Figure 36). The purpose of the Landslip Warning is to alert the public to take appropriate action to reduce their exposure to possible danger from landslides (Chan & Pun 2004). The issuing of the Landslip Warning also triggers an emergency system within government departments that mobilizes staff and other resources to deal with landslide incidents. Based on landslide records in the past 20 years, Landslip Warnings have proved to be reliable with over 90% of the landslide fatalities happening at the time of the Warnings.
KRA 7: Enhance the Appearance and Aesthetics of Engineered Slopes

Meeting Community’s Environmental Needs
In the past 30 years, there have been continual changes in public expectations on slope safety. Up to the early 1990’s, public attention was mainly focused on safety with constant demand on “speedy” reduction of landslide risk. The approach adopted was to provide hard cover (e.g. shotcrete) to cut slopes and grass cover for fill slopes; emergency works following landslides were shotcreted to restore the temporary stability of the failed slopes after clearing of the failure debris and re-grading of the steep portions. With substantial improvement in slope safety and increase in environmental awareness of the built environment, it is now government’s policy to make engineered slopes look as natural as possible. Every effort is therefore made to green slopes by hydroseeding and planting trees and shrubs (Figure 37). Where slope greening is not feasible either on safety grounds (e.g. during emergency) or due to steep topography, appropriate landscaping works are incorporated to enable the slope to blend with the environment (Figure 38).

Figure 36: Landslip Warning signal

Figure 37: Greening and landscaping of man-made slopes

Figure 38: Landscaping works for slope with hard cover
For rectifying loose fill slopes, which are commonly covered by mature trees and vegetations, re-compaction of the fill will entail removal of trees and strong public objections. As discussed in KRA 3, the development of the soil-nailing method for stabilizing loose fill slopes has greatly helped to meet public expectation of tree preservation (Figure 15).

It is of interest to note that not all greening efforts are supported by local residents who may raise concern on a greater need for maintenance or source of mosquitoes and insects as compared to hard cover, i.e. “not-in-my-backyard” attitude. Consultation with nearby residents has been a norm to ensure that the proposed slope cover meets their expectations.

**PERFORMANCE OF THE HONG KONG SLOPE SAFETY SYSTEM**

In evaluating the performance of the System, several approaches have been adopted:
(a) quantitative assessment of the reduction in landslide risk;
(b) public opinion polls on key elements of the slope safety issue;
(c) success rate in preventing major landslides; and
(d) trend of fatalities resulting from landslides.

**Quantitative Assessment of Landslide Risk**

Using QRA techniques, GEO has assessed the risk to the community due to landslides. Results show that in year 2000, the overall landslide risk arising from old substandard man-made slopes has been reduced to less than 50% of the 1977 level when the GEO was established. Taking into account the progress of LPM works from year 2000, it is estimated that the overall landslide risk will be further reduced to 25% of the 1977 level. Details of the assessments are given in (Chan et al. 2007b).

**Public Opinion Polls on Slope Safety**

Since 1997, GEO has engaged local universities to conduct independent public opinion polls on how the community views key elements of the slope safety issue. Latest results in the 2006 public opinion survey are given in Table 1.

Table 1: Results of public opinion survey in 2006

<table>
<thead>
<tr>
<th>Results of Public Opinion Survey in 2006</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level of public awareness of slope safety problem</td>
<td>85%</td>
</tr>
<tr>
<td>Slope owners’ understanding of their maintenance responsibility</td>
<td>71%</td>
</tr>
<tr>
<td>Public awareness of personal safety precautions during rainstorms</td>
<td>79%</td>
</tr>
<tr>
<td>Public understanding of the importance of slope maintenance</td>
<td>94%</td>
</tr>
<tr>
<td>Level of public satisfaction about slope appearance</td>
<td>69%</td>
</tr>
</tbody>
</table>

Public opinion survey results in the past 10 years (Figure 39) confirm our belief that the public is supportive and appreciative of government’s efforts on slope safety.
Success Rate in Preventing Major Landslides
Under existing arrangements, GEO regularly receives information on landslides in Hong Kong. A Landslide Database has been set up containing pertinent information of landslides. In the Database, landslides exceeding 50 m³ have been classified as major landslides. Annual review of the landslides during the year has been ongoing for over 10 years. Year 2005 and 2006 were respectively a wet and dry year. The performance indicators for these two years are given in Table 2 below. The results give a clear indication that the overall performance of the Slope Safety System is good.

Table 2: Success rates in preventing major landslides

<table>
<thead>
<tr>
<th>Performance Indicator</th>
<th>2005 (Wet Year)</th>
<th>2006 (Dry Year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Success rate (the percentage of audited slopes which performed satisfactorily during the year) in preventing major landslides in slopes audited as conforming to the required safety standards</td>
<td>99.98%</td>
<td>100%</td>
</tr>
<tr>
<td>Success rate (the percentage of upgraded government slopes which performed satisfactorily during the year) in preventing major landslides in upgraded government slopes</td>
<td>100%</td>
<td>100%</td>
</tr>
</tbody>
</table>

Trend of Landslide Fatalities
Although the fatality rate due to landslides is not a true indicator of landslide risk, the trend of the landslide fatalities does give some indication of the reduction in landslide risk with time due to the slope safety works carried out in Hong Kong for the past 30 years. Figure 40 shows that there is a drastic reduction in the landslide fatality rate since the establishment of the GEO in 1977. It is evident that the reduction of landslide risk is very significant.
WAY FORWARD
On completion of the current phase of the LPM Programme in 2010, it is expected that all high-risk man-made slopes (i.e. old substandard slopes affecting buildings and busy roads) will have been retrofitted with the landslide risk reduced to 25% of the 1977 level. The remaining problem is to contain landslide risk within this low level. Continuing efforts are required to:

(1) prevent increase in landslide risk due to slope degradation and urban development; and
(2) discharge Government’s due diligence in dealing with known landslide hazards.

The majority of the remaining landslide risk comes from two main groups of slopes, viz. man-made slopes with moderate risk and natural hillside catchments with known history of landslides close to buildings and important transport corridors.

For man-made slopes, as opposed to the ‘Total Retrofit’ approach currently adopted, an ‘Asset Management’ approach is being worked out to identify those moderate-risk slopes that are at a more advanced state of deterioration for prompt follow-up action, on a rolling basis.

For natural terrain, risk mitigation works will be systematically implemented on natural hillside catchments with known hazards, in accordance with the ‘react-to-known-hazard’ approach.

Together with the ongoing efforts of controlling the standard of new slopes through geotechnical control, regularly maintaining government slopes and promoting public awareness and response in slope safety through public education, publicity, information services and public warnings, the Hong Kong Slope Safety System will ensure a more sustainable slope safety environment and meet public expectation in slope safety. The
post-2010 Landslip Prevention and Mitigation Programme is being worked out to dovetail with the current Landslip Preventive Measures Programme.

CONCLUSIONS
Hong Kong is characterized by dense urban developments on or near to steep hillsides. Coupled with the torrential summer rainfall, the challenges that Hong Kong has been facing with regard to slope safety are unique. Our comprehensive Slope Safety System has won high regard from the geotechnical community both locally and internationally. The key elements of the Hong Kong Slope Safety System include improving slope safety standards and technology, ensuring the safety of new slopes, improving the safety of existing slopes, enhancing slope appearance, promoting proper maintenance of slopes and provide public education, publicity, and public warnings as well as information services on slope safety. Given the geographical and climatic setting of Hong Kong, landslide risk could never be reduced to zero. However, through the concerted efforts of government and the support and participation of the community, it is encouraging to see substantial progress towards meeting Hong Kong’s needs for the highest standards of slope safety.

REFERENCES
Buildings Department (BD) (2006). Division of Responsibilities between Authorized Person, Registered Structural Engineer and Registered Geotechnical Engineer, PNAP 294, Buildings Department (BD), Hong Kong, 16 p.
Chan, R. K. S., Mak, S. H., and Au Yeung, Y. S. (2007a). “Partnering with the community to reduce landslide risk in Hong Kong over the past thirty years.” Proceedings of 27th HKIE Geotechnical Division Annual Seminar, Geotechnical Advancement in Hong Kong since 1970s, Hong Kong Institution of Engineers (HKIE), May 2007.


Hong Kong Institution of Engineers (HKIE) (2003). *Soil nail in loose fill slopes – A preliminary study. Final Report.* Geotechnical Division of the Hong Kong Institution of Engineers (HKIE), Hong Kong, 47 p.

Ho, K. K. S., and Pappin, J. W. (2007). “Geotechnical failures in Hong Kong.” *Proceedings of 27th HKIE Geotechnical Division Annual Seminar; Geotechnical Advancement in Hong Kong since 1970s,* Hong Kong Institution of Engineers (HKIE), Hong Kong, 213-224.


EXPERIENCE ON LANDSLIDE RISK MANAGEMENT IN THE EASTERN PYRENEES (SPAIN AND ANDORRA): ACHIEVEMENTS AND CHALLENGES

Jordi Corominas
Department of Geotechnical Engineering and Geosciences
Technical University of Catalonia, Barcelona, Spain

Abstract: This communication reviews the experience of both the Catalonia autonomous region (Northeast Spain) and the Principality of Andorra in landslide hazard assessment and risk management in the Eastern Pyrenees during the last 20 years. The risk management strategies of both regions have been conditioned by the amount of available land and the pressure for development. In the Eastern Pyrenees, landslide hazard maps at medium scale are available since the mid-late 80s. The strategy for risk management of the Catalonia region has been mostly based on land-use planning measures. However, the region still lacks of an official landslide map and consequently, policies to restrict the development can not be fully implemented. In the Principality of Andorra, building restriction initiatives have been combined with protection measures, particularly for rockfalls. The most important initiative has been the Rockfall Risk Management Master Plan of the Solà d’Andorra. In the year 2000, a new landslide hazard map at 1:5,000 scale was prepared which has become the basic document for the implementation of building codes and land use regulations. The Andorran administration is currently engaged in an ambitious programme for landslide risk mitigation with special interest in both the urban areas and the main road network. The principles of the landslide hazard maps, the risk management measures and the future challenges, are discussed in the paper.

INTRODUCTION

From an economical viewpoint, landslides are the third most important type of natural hazards in Spain, after floods and earthquakes. The expected losses due to landslides for the period 1986-2016, for the most probable scenario, rise up to more than 4500 millions euros with a mean annual cost exceeding 200 million euros (Ayala-Carcedo et al. 1987). However, quite often damages attributed to floods and earthquakes are caused by landslides and consequently, these figures do not reflect the overall significance of the landslide hazard and should be taken as a minimum estimation.

In Spain, landslides occur preferably in the mountainous ranges, the slopes of the Tertiary basins and the coastal cliffs. The Pyrenean cordillera is the area most affected by landsliding. It has more than 350 km in length. From a geological point of view, two zones are clearly distinguishable: the basement that outcrops is the so called axial zone and the cover that forms the outer ranges at both northern and southern sides of the range. The basement consists almost entirely of igneous and metamorphic Paleozoic rocks folded, intruded and metamorphosed during the Hercinian orogeny. The cover is composed of sedimentary sequences mostly of Mesozoic ages (Figure 1).
The axial zones of the Central-Eastern Pyrenees form an antiformal-shape thrusts stack bounded by an imbricated thrust system (Muñoz 1992). The Central-Eastern Pyrenees are divided into Northern Pyrenees and Southern Pyrenees. The south-Pyrenean is formed by several thrust sheets grouped into two main structural units: the Upper and Lower Thrust Sheets, which are formed by imbricated thrust systems. The Upper Thrust Sheets consist of cover rocks, mainly Mesozoic in age. The Lower Thrust Sheets involve Hercynian basement and cover rocks. A major fault, called the North Pyrenean Fault, separates this structural unit from the Northern Pyrenees.

Most of the Pyrenees was covered by Pleistocene glaciers. Data on the glaciations indicate the last maximum stage around 50,000 years BP, while the principal glacier retreat started between 30,000 and 20,000 years BP. The glaciers attained a maximum length of up to 50 km, a maximum thickness of about 900 m, and they reached to a minimum elevation of about 700 - 900 m a.s.l.

In this communication, I will review the experience in landslide risk management during the last twenty years in the Southern-Eastern Pyrenees. This part of the Pyrenees has a length of more than 150 km and is managed by the Catalonia autonomous government and the Andorra Principality. The Catalonian autonomous region is located at the northeast of Spain (Figure 2), it has an extension of almost 32,000 km² and a population of over 7 million inhabitants. The Principality of Andorra is a tiny country located in the heart of the Pyrenees between France and Spain. It has an extension of 468 km² with a mean height of 1900 m. a.s.l. and a population of over 75,000 persons. Both Catalonia and Andorra have been historically affected by a diversity of hazardous phenomena, in particular, floods, landslides, and snow avalanches.
OCCURRENCE OF LANDSLIDES IN THE EASTERN PYRENEES

Landslides are local phenomena and the statistics of their occurrence and consequences are scarce. Table 1 includes a list of main events recorded in Spanish Eastern Pyrenees and the Principality of Andorra. Even though this table might not be complete, it shows an increasing trend in the number of both victims and the economic damages. This increase is in part due to the greatest interest for the landslide phenomena but also to the more intensive use of the mountain areas. The tourist-related activities and both winter and summer sports have spread over the slopes and have generated an influx of visitors never seen before in places where rockfalls and slides are frequent (Figure 3). The exposure of the persons, settlements and infrastructures has increased and consequently, the number of the incidents has risen as well.
This greater human presence also increases the instability of the slopes. Research studies carried out in the Cantabria region (Remondo et al. 2005) have shown that between 1954 and 1997, the frequency and volume of the slides was multiplied by a factor of ten without significant changes in the total rainfall, the number of storms or the annual number of rainy days above certain thresholds. Instead, a relation does appear between the landslide occurrence and the degree of human intervention in the territory.

Figure 3: Landslide at L’Estartit reactivated during the rainfall events of October and November 1994

CAUSES OF THE SLOPE INSTABILITY IN THE EASTERN PYRENEES
The spatial distribution of landslides in the Eastern Pyrenees is governed by the relief, the presence of susceptible materials and the structural setting. The influence of the relief is enhanced in the formerly glaciated valleys by the presence of steep slopes and by the erosive action of the present fluvial network. The outcrop of susceptible materials is critical for the development of the slope failures (Table 2). Several lithostratigraphic formations are very susceptible to the instability phenomena. A synthesis is found in Corominas and Alonso (1984). Among the different lithologies, Silurian shales are associated with large slope failures such as in Pardines (Girona) and Pont de Bar (Lleida), which are mainly earthflows but also translational slides developed through the weakest clayey layers (Bru et al. 1984a). Likewise marls and gypsum layers of Triassic (Keuper) age are often affected by rotational slides and earthflows as in Pont de Suert. Mesozoic flysch facies produce complex failures: rotational and earthflows or slides through existing layers. Glacier deposits (tills) are affected by debris flows, debris avalanches and rotational slides (Bru et al. 1984b). Failures in these materials have left distinct scars at La Guingueta, Artíes, Taüll, Capdella or Bono (Lleida), Senet and Benasque (Huesca). Colluvial deposits, which are frequent in slopes of less than 45º, show frequent shallow slides and debris flows.
Table 1: Main landslide events in the Spain and Andorra (the Eastern Pyrenees) during the last 150 years with respect to victims and relevant damages

<table>
<thead>
<tr>
<th>Locality (Province)</th>
<th>Date</th>
<th>Type of failure</th>
<th>Consequences</th>
</tr>
</thead>
<tbody>
<tr>
<td>Les Escaldes (Andorra)</td>
<td>April 16th, 1865</td>
<td>Debris flow – debris avalanche</td>
<td>El Fener neighborhood destroyed</td>
</tr>
<tr>
<td>Puigcercós (Lleida)</td>
<td>January 13th, 1881</td>
<td>Translational slide</td>
<td>Houses destroyed. Village abandoned</td>
</tr>
<tr>
<td>Bono (Lleida)</td>
<td>October 26th, 1937</td>
<td>Debris avalanche</td>
<td>River dammed</td>
</tr>
<tr>
<td>Gran Valira valley (Andorra)</td>
<td>October 1937</td>
<td>Debris flows</td>
<td>Widespread debris flow activity</td>
</tr>
<tr>
<td>Eastern Pyrenees</td>
<td>October 18th, 1940</td>
<td>Hundred of shallow slides</td>
<td>Several affected roads and infrastructures</td>
</tr>
<tr>
<td>Rocabruna (Girona)</td>
<td>October 18th, 1940</td>
<td>Debris flow</td>
<td>6 casualties</td>
</tr>
<tr>
<td>Senet, Benasque (Huesca), Arties (Lleida)</td>
<td>August 3rd, 1963</td>
<td>Debris flow</td>
<td>River dammed. Roads affected</td>
</tr>
<tr>
<td>Eastern Pyrenees</td>
<td>November 7th, 1982</td>
<td>Thousand of shallow slides and large landslide reactivation</td>
<td>Several affected roads, villages and infrastructures</td>
</tr>
<tr>
<td>Pont de Bar (Lleida)</td>
<td>November 7th, 1982</td>
<td>Slide</td>
<td>Houses destroyed. Village abandoned</td>
</tr>
<tr>
<td>Capdella (Lleida)</td>
<td>November 7th, 1982</td>
<td>Debris flow</td>
<td>Two houses destroyed; 4 casualties</td>
</tr>
<tr>
<td>Guixers (Lleida)</td>
<td>October 1987</td>
<td>Rockfall</td>
<td>2 deads. Car hit</td>
</tr>
<tr>
<td>La Massana (Andorra)</td>
<td>October, 10th 1987</td>
<td>Rock Slide</td>
<td>3 casualties and 2 persons injured. Several cars hit</td>
</tr>
<tr>
<td>Camprodón (Girona)</td>
<td>May 1992</td>
<td>Debris flow</td>
<td>2 casualties</td>
</tr>
<tr>
<td>L’Estartit (Girona)</td>
<td>October-November 1994</td>
<td>Slide</td>
<td>Several houses destroyed</td>
</tr>
<tr>
<td>Pont de Suert (Lleida)</td>
<td>November 1994</td>
<td>Slide-earthflow</td>
<td>Road affected</td>
</tr>
<tr>
<td>Gerri de la Sal (Lleida)</td>
<td>November 10th, 1994</td>
<td>Rockfall</td>
<td>1 casualty and 1 injured</td>
</tr>
<tr>
<td>Santa Coloma (Andorra)</td>
<td>January, 1997</td>
<td>Rockfall</td>
<td>1 casualty and building damaged</td>
</tr>
<tr>
<td>Sant Corneli (Barcelona)</td>
<td>December 17th, 1997</td>
<td>Rotational slide</td>
<td>1 injured. Road cut. 1 million euro damages</td>
</tr>
<tr>
<td>Cala Sr. Ramon de Palafrugell - Girona</td>
<td>August 25th, 2003</td>
<td>Rockfall</td>
<td>2 casualties and 2 injured</td>
</tr>
<tr>
<td>Barruera – Vall de Boí-Lleida</td>
<td>September 20th, 2003</td>
<td>Rockfall</td>
<td>2 injured. Road cut</td>
</tr>
</tbody>
</table>

Table 2: Unstable lithologies in the Eastern Pyrenees (Spain and Principality of Andorra) and the associated types of failure (synthesis based on data from Corominas 1989)

<table>
<thead>
<tr>
<th>Formation</th>
<th>Age</th>
<th>Type of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Black shales</td>
<td>Silurian</td>
<td>Slides, earthflows</td>
</tr>
<tr>
<td>Claystones and gypsum</td>
<td>Keuper</td>
<td>Rotational and translational slides, earthflows</td>
</tr>
<tr>
<td>Alternances of lutites, sandstones, lignites (Facies Garum)</td>
<td>Upper Cretaceous</td>
<td>Rotational and translational slides and earthflow</td>
</tr>
<tr>
<td>Marly clays</td>
<td>Lower Eocene – Lutecian Eocene inferior</td>
<td>Rotational slides and earthflow Earthflow, slides</td>
</tr>
<tr>
<td>Marls and alternances of sandstones, marls and limestones (Flysch)</td>
<td>Miocene</td>
<td>Rotational slides and mudslides Debris flows and avalanches. Rotational slides</td>
</tr>
<tr>
<td>Clays, sandy silt</td>
<td>Pleistocene</td>
<td>Slides and debris flow Large slides Rockfalls</td>
</tr>
<tr>
<td>Boulders and gravels with sandy-silty or clayey matrix (glacial till)</td>
<td>Miocene-Holocene, Pleistocene, Pliocene, Pleistocene</td>
<td></td>
</tr>
</tbody>
</table>
The structural arrangement of the Pyrenean range also conditions the appearance of slope failures. Even in resistant rocky formations, instability may be produced by the presence of structural weaknesses (bedding planes, joints, faults, schistosity planes). When the latter is dipping unfavourably in relation to the orientation of the slope, large slides can develop in calcareous formations, in granites or in sandstones. This is particularly evident in the sedimentary formations in the pre-Pyrenees. This accounts for the large translational slides like the one in Vallcebre (Corominas et al. 1999).

Figure 4: Source area of debris flows in the till deposits located above the village of Taüll

Figure 5: Translational slides developed on the bedding surfaces of the Mesozoic limestones in the Coll de Pal road
Several triggers may account for the occurrence of landslides in the Eastern Pyrenees: rainfall, earthquakes, river erosion, although the influence of the human activities as either conditioning or triggering factor is increasing rapidly and should not be disregarded. In any case, the vast majority of the Pyrenean landslides are caused by rainfalls. The Eastern Pyrenees is the part of range subjected to the influence of the Mediterranean climate which is characterized by dry summers and intense rainfall episodes in autumn. In the last century, the biggest episodes of river flooding and slope instability in the Eastern Pyrenees were fundamentally concentrated between October and November although there have been sporadic episodes distributed throughout the other seasons. Of particular significance were the events in October 1907 and 1937 in the upper Segre basin, October 1940 in the Ter basin, November 1982 in the basins of the rivers Llobregat, Segre and Noguera Ribagorçana. Three rainfall patterns have been distinguished that cause slope failure and the reactivation of slides (Corominas 2000; Corominas et al. 2002): (a) short duration high-intensity rainfall; (b) rainfall episodes of moderate to low intensity that last for several days or weeks; and (c) abnormally rainy seasonal and interannual episodes.

High intensity and short-lasting rains are able to trigger shallow landslides, and debris flows. These types of failure take place mainly in slopes covered with pervious materials. The rainfall infiltrates rapidly into the soil through voids and micropores, causing the build up of pore water pressures that lead to the slope failure. The analysis of the isohyets and of the areal extent of the shallow slope failures in several landsliding events has allowed the establishment of a rain threshold of 190 mm in 24-36 hours for the Eastern Pyrenees (Gallart & Clotet 1988; Corominas & Moya 1999). In cases analyzed, no significant rainfall was recorded during the weeks prior to the occurrence of the failures.

Low to moderate intensity rainy episodes are able to reactivate mudslides (earthflows) and both medium-sized rotational and translational slides in low permeability soils, provided that antecedent wetness is available in the ground. In the Eastern Pyrenees, movements with volumes of some tens to few hundreds of thousand of cubic metres, are reactivated by moderate rainfall episodes of 40 to 100 mm in 24 hours if, at least, 90 mm of antecedent rain have fallen during the previous days (Corominas & Moya 1999). These types of landslides usually take place on clayey and silty-clayey geological formations which are of low permeability. In such formations, water infiltration from the surface is restricted by the hydraulic conductivity of the materials, which is mostly controlled by the grain size and, to a lesser extent, by soil fissures and recharge through pervious layers such as interbedded sandstone layers. These episodes usually do not cause failures in colluvium because of the presence of large interparticle voids and macropores (animal burrows, decayed root channels, soil pipes) that drain water infiltrated from moderate-intensity rains very rapidly.

Abnormally rainy seasons cause the reactivation of large-scale slides. In particular geological contexts, short duration rainfall can also cause reactivation of large landslides. The behavior of large landslides still has many unknowns. Historical records show that most first-time failures in large landslides were caused by non-climatic factors (Corominas 2000). However, rainfall is the most frequent cause of reactivation of dormant slides and of the acceleration of those that are already active. It is not easy to establish the relationship between rainfall and the activity of large landslides; this is due to the fact that we do not yet avail of sufficient knowledge of their hydrological behavior. Our experience studying recent landslides in the Eastern Pyrenees has shown that the relationship between rainfall and occurrence of large landslides is complex (Corominas 2000). In general, long rainy periods (at seasonal, annual or ten-year scale) appear to have a certain influence in the reactivation of large landslides.
although the relationship can often only be established in a qualitative manner. In very particular geomorphological contexts that favor instability, either through extraordinary amounts of groundwater (e.g. contact with karstic massifs) or due to brusque topographic changes (e.g. toe erosion), large landslides can be reactivated by intense and short-lived rainfall episodes. Some cases were observed during the intense rainfall on November 6th-7th, 1982 in the Eastern Pyrenees (Corominas & Alonso 1990).

**ASSESSING LANDSLIDE HAZARD IN THE EASTERN PYRENEES**

Spain is organized in autonomous regions. The region of Catalonia has authority over regional planning and land development, and on civil protection. In what concerns, the landslide risk management, the Catalanian autonomous administration and the Principality of Andorra have followed different strategies (see Figures 6 and 7). These strategies have been conditioned by the availability of land, the pressure for development and the socioeconomic impact of the previous landsliding events. In the Catalan Pyrenees, land use planning measures have been adopted with a minimum of structural measures. Instead, the lack of available land in Andorra has led the administration towards the development of building codes and the implementation of protective measures.

The awareness of the threat caused by landslides was enhanced by the November 1982 event. On this date, the Eastern Pyrenees were affected by heavy rains lasting for 36 hours with catastrophic consequences. The recorded rains ranged between 80 and 555 mm, with a maximum daily record of 342 mm at La Molina (Novoa 1984). These rains caused extensive flooding and thousands of landslides which resulted in a death toll of 19 lives and about 600 million of euros in direct damages.

An indispensable requisite for the appropriate landslide risk management is the availability of both landslide inventories and maps. Catalonia region and the Principality of Andorra have promoted landslide hazard maps. The 1982 catastrophic event became a benchmark for the hazard assessment studies. In 1985, the Department of Public Works and Urban Planning of the Catalanian government commissioned the preparation of natural hazard maps for the Eastern Pyrenees. Ten counties were mapped at 1:50,000 scale (Corominas 1985). The phenomena analyzed were landslides, snow avalanches, floods and sinkholes and the methodology followed a heuristic approach. The different hazardous phenomena were identified and located using aerial photographs, and checked with field work. Concerning the landslides, four hazard levels were established based on the presence of large active movements, large dormant movements, shallow landslides, and areas where instability processes have not been identified (Figure 8). Due to the working scale, only large landslides or long landslide tracks were represented with their real boundaries. Most of the landslide phenomena were plotted with areal symbols (i.e. area affected by shallow landsliding). In 2002, the Catalanian Geological Institute (CGI) started updating these maps, which have been renamed as county maps for the prevention of the geological hazards. The new generation of maps include the same processes plus the basic seismic acceleration. At present, 13 counties have been completed. In 2007, the CGI has started an ambitious plan of landslide hazard mapping for the whole Catalonia at 1: 25,000 scale that is expected to be completed by 2012.
Figure 6: Relevant landsliding events and action undertaken by the Catalonian autonomous administration

Figure 7: Relevant landsliding events and action undertaken by the Andorra government
The efforts of the Andorran administration in natural hazards management began in the early 1980s. In June 1980, the Consell General (Government Council of Andorra) promoted a hazard regulation for building in places threatened by snow avalanches, rock falls, and torrential activity. The resolution foresaw the preparation of an inventory of hazards and the possibility of the suspension of building permits in the identified threatened sites. However, the building restriction was seldom put into practice because the comprehensive hazard inventory was completed only for the snow avalanche phenomenon. The rainy event of November 1982 struck the heart of the country. The water of the Gran Valira river overflowed the capital Andorra la Vella and Les Escaldes and invaded the main commercial streets and roads. The overall toll was 9 people dead and 4 people missing and more than 55 million euros in material losses. In October 1987, rains lasting for several days triggered a landslide in a quarry located next to the road to La Massana. About 50,000 m$^3$ of rock slid down and hit several cars. Three people died in this episode and the Valira del Nord valley and all its villages remained isolated for several weeks.

The first global initiative in the domain of the landslide hazard assessment and prevention was the natural hazards maps at 1:25,000 scale, that included landslides, torrential floods and flood-prone areas. In 1989, the first sheet covering the Valira d'Orient and Gran Valira valleys was completed, and that of Valira del Nord was completed in 1991 (Corominas et al. 1990). In the year 2000, a new landslide hazard map at 1:5,000 scale was prepared which has become a basic document for the development of building codes and land use regulations. Private developments must set up the necessary protective measures in order to obtain building permits in the threatened areas. The map has given way to more detailed studies at 1:2,000 scale in the most conflicting areas of the Principality and to the execution of remediation projects and development of strategies for living with risk. The principles on which the above mentioned hazard maps are based on, are discussed later.
Figure 9: Partial view of the rockslide of La Massana, triggered in October 1987. Several cars were trapped and three people died. The Valira del Nord Valley was blocked for more than one month.

LANDSLIDE RISK MANAGEMENT: LAND USE PLANNING MEASURES
The Catalonia region has no administrative departments specifically in charge of the natural disasters. The river flood risk management is carried out by the river basin authority (Catalonia Basin Authority) but no particular unit is in charge of the management of landslide hazard. In the 1980s, there was no specific ordinance in relation to natural hazards in Spain. The existing Land Use law of 1975 only stated that the local development plans should include studies for the characterization of natural environment of the territory. The natural hazard maps of the mountain counties prepared in 1985 were added to the information used to prepare the county land use plans although they were purely informative, but not legally binding documents. Because of this, no specific recommendations related to natural hazards were included in these plans. A new Land Use Law in 1998 established that land potentially at risk must not be developed. The condition of land at risk had to be accredited by a natural hazard map and plan; however this has not been done yet for landslide zoning. Instead, flood prone areas were included in the Basin Hydrological Plans and flood catalogues were prepared as well. The Spanish Civil Protection Directive of 1994 established that hazard
maps for floods, earthquakes and volcanoes had to be prepared for the whole Spain although no mention was made again to the landslides. In 1995, the Civil Protection Plan for Catalonia (PROCICAT) was prepared. This Plan foresaw the preparation of Special Plans for floods (INUNCAT) and earthquakes (SISMICAT). The INUNCAT plan was approved in 2006 and even though the main goal is river flooding, it takes landslides into account as associated phenomena. The Plan includes an inventory of debris fans, landslides and locations where slopes may be undermined by rivers. The catalan territory is divided in four classes: (a) areas with mid to high landslide hazard where detailed studies must be performed in the local development plans; (b) areas of low hazard where detailed studies are not required; (c) areas where landslide may occur due to slope undermining by river erosion; and (d) low landslide hazard areas located in flood-prone areas.

![Map of flooding areas and debris fans (INUNCAT 2006)](image)

Figure 10: Map of flooding areas and debris fans (INUNCAT 2006)

In 2005, the Catalonian regional administration approved a new Urban Planning Law establishing that land development must provide the adequate level of protection to the population. This law states that the development should be avoided in areas threatened by natural hazards if countermeasures are not feasible. Furthermore, the law states that urban settlements can not develop on slopes over 20% unless no alternative land exist.

The assessment of both natural and geological hazards for urban planning purposes must take into account the official maps of the Catalonian Geological Institute (CGI). All the local development plans are nowadays sent to the CGI for providing reports on the potential hazards. These reports consider specifically all kind of natural hazards and provide recommendations to the municipalities. Since year 2000, more than 250 reports have been issued, which represents approximately 25% of the municipalities in Catalonia. At present Catalonia lacks of official landslide maps to depict at an appropriate scale the landslide hazardous areas. Because of this, the reports issued by the CGI are not legally binding. As a consequence, the policies to restrict the development in the landslide threatened areas cannot be fully implemented and depend largely on the interest and proactive behaviour of the local
communities. This situation will change in few years once the plan for mapping geological hazards at 1:25,000 scale had been completed.

Significant differences are found in the treatment given to the flood hazard and to the landslide hazard in the Urban Planning Law. First of all, floods are specifically considered as a type of natural hazards while landslides are included within the generic term of natural and geological hazards. Secondly, the fluvial space is explicitly divided in three zones: the fluvial zone, the hydric system zone, and the zone affected by extraordinary floods. The fluvial zone is defined by the watercourse and the part of the flood plain affected by the 10 year-return period flood. The urban planning must consider this zone as part of the hydraulic system and no development is allowed except those infrastructures related to the river system. The hydric system zone is defined by the extent of the 100 year-return period flood. In this zone, new buildings or constructions are not allowed and also activities that could modify the natural profile of the ground, act as obstacles to the water flow, or alter the flow regime in case of flood. The agricultural and leisure activities, pumping stations and water treatment plants, transport and communication infrastructures are allowed, provided that they will preserve the river flow regime. The zone affected by extraordinary floods is defined by the extent of the 500 year-return period flood. In this zone, development is restricted in different degrees depending on whether it is affected by a slight, moderate, or serious level of flooding. Unfortunately, the decree developing the Urban Planning law does not introduce land-use recommendations based on the magnitude (intensity), return period, and landslide hazard levels.

In the Principality of Andorra, the natural hazard maps prepared in 1990 and 1991 were used by the administration to restrict the development in the most hazardous areas. These maps, however, were not official documents and not legally binding.

The main step in management of the natural hazards was given by the Urban and Land-Use Planning Law approved in 1998. The key points of this law in terms of hazard management are the following (Escalé 2001): (a) zones exposed to natural hazard cannot be developed; (b) local development plans must take into account the presence of zones exposed to natural hazards; (c) the Andorra government will commission both geological-geotechnical studies and hazard mapping. This means that the Andorra government has to provide hazard inventories, hazard zoning and regulations for management of the threatened areas. In those sites where hazard can be mitigated and reduced to an acceptable level, the Andorrain government will establish the requirements of the protective works that have to be undertaken.

After the implementation of the law, several studies have been completed: the snow avalanche hazard mapping (1988-1999), the study of the flood hazard and the proposed protective measures for the whole river network of Andorra (1999-2001), and the Geotechnical and Landslide Hazard Zoning Plan of Andorra (1999-2001).

The Geotechnical and Landslide Hazard Zoning Plan of Andorra
The purpose of the geotechnical and Landslide Hazard Zoning Plan of Andorra (hereafter refer to Geohazard Plan) was to identify, locate and assess the natural hazards along with the geological and geotechnical constraints that may affect future construction works in the Andorran territory. The scale of work was 1:5,000, which has enough detail to identify most of the existing and potential hazards but it does not allow a proper definition of the landslide boundaries (both source and runout area) for cadastral purposes. On December 2001, the Plan
was published in the official registrar of Andorra (BOPA) and public audience and amendment period was open until February 2002.

The Geohazard Plan is based on the landslide hazard map prepared at 1:5,000 scale. The preparation of the map involved several steps (Corominas et al. 2003a): the assessment of the potential slope failures and the estimation of both the landslide volume and runout distance. In the identified susceptible areas, landslide magnitude and frequency was determined in order to obtain the hazard zoning map. Data required for hazard assessment were introduced into a GIS or derived directly from available Digital Terrain Models. The assessment of intensity and frequency for all landslide types was carried out in a GIS at each susceptible cell. As a result all cells were classified according to a hazard matrix (Table 3) and then the resultant hazard zoning map was prepared following criteria similar to those used elsewhere (Lateltin 1997).

Table 3: Landslide hazard categories for rockfalls

<table>
<thead>
<tr>
<th>Frequency (return periods)</th>
<th>&lt; 40 yr</th>
<th>40-500 yr</th>
<th>&gt; 500 yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>High &gt; 10000 kJ</td>
<td>High</td>
<td>High</td>
<td>Moderate</td>
</tr>
<tr>
<td>Medium</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>Low &lt; 2000 kJ</td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Non-susceptible areas</td>
<td>Very low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
</tbody>
</table>

The map (Figure 11) has four hazard categories: (a) very low, in which no potential hazard has been observed; (b) low, in areas that may be affected by small-sized slope failures with moderate-high frequency and that can be mitigated at low cost; (c) moderate, corresponding to areas where either frequent landslides of small magnitude or large landslides with low frequency may take place. Landslide countermeasures are feasible; and (d) high hazard is assigned to areas where large landslides may reactivate or are active. Landslide countermeasures are not feasible.

One of the recommendations of the Geohazard Plan was the performance of more detailed studies in the areas subjected to the greatest urban pressure and in which high or moderate hazard was obtained.
An administrative procedure has been established for delivering building permits taking into account the hazard classes defined in the Landslide Hazard Map (Corominas et al. 2003b). Different documents may be asked for new developments or infrastructures according to the degree of hazardness of the site, which are synthesized in Table 4. For areas classified as very low landslide hazard, no specific document is required. For low hazard areas, the owner or developer must fill in a form of acknowledgement of the type of threat that may affect the property. This form is signed by the engineer or architect responsible of the project, mentioning that the possible hazard has been taken into account for the project design. For moderate hazard areas, besides the acknowledgement form, a technical report is required. This report must include specifically the countermeasures that will be undertaken in order to avoid or mitigate the potential hazard along with an estimation of the residual risk (particularly for those events of large return periods). In this hazard category, sensitive buildings such as schools or hospitals are not allowed. Finally, for high hazardous areas new constructions or facilities are forbidden. A few exceptions are, however, envisaged. Warehouses with no permanent activity, linear infrastructures (i.e. water pipes) that will not threat population or the environment in case of failure, or roads without alternative corridors might be allowed and, in this case, both acknowledgement form and technical document will be required to justify the project technically.

Figure 11: Landslide hazard map of the Encamp area at 1:5,000 scale, obtained by overlapping of rock fall, shallow slide, debris flow and large landslide hazard maps. In case of coincidence of different landslide types in the same site, the highest hazard class is shown.
### Table 4: Administrative procedure for delivering building permits

<table>
<thead>
<tr>
<th>Hazard Category</th>
<th>Documents required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>None</td>
</tr>
<tr>
<td>Low</td>
<td>Acknowledgement form (AF)</td>
</tr>
<tr>
<td>Moderate</td>
<td>AF + Technical report (TR)</td>
</tr>
<tr>
<td>High</td>
<td>Building forbidden</td>
</tr>
<tr>
<td></td>
<td>Exceptions: AF + TR</td>
</tr>
</tbody>
</table>

The procedure lets open the possibility of authorization to build in high hazardous areas if the promoter demonstrate with the adequate technical studies that countermeasures to avoid or mitigate instability are feasible. The hazard map is integrated in a GIS, thus allowing the knowledge of the type of hazard that threatens a particular site. Therefore, developers of this site may know in advance what kind of technical report they will be asked for. The Administration has made the population conscious of the existing level of hazard by informing the municipalities, promoting open informative sessions and by publishing it as a decree in the official journal in August 2001. By asking the AF and TR, the Administration guarantees that the potential hazard has been taken into account and that either protective or remedial measures will be implemented. In case of completion of the TR and the subsequent protective works, the Administration will deliver the corresponding building permits.

The landslide hazard map at 1:5,000 scale, showed that some areas subjected to an intense urban pressure are considered of a moderate hazard. Most of these areas correspond to either large dormant or slow moving landslides or debris fans with a defined debris source located upstream. The characteristics of such landslides make the completion of the AFs and TRs too complex and costly for the private owners. In order to speed up the whole procedure and to avoid unnecessary delays in the urban development of the Principality, the Ministry of Public Works commissioned detailed studies at several landslide sites that have required further landslide hazard analyses (Hürlimann et al. 2006). Each study included landslide hazard maps at 1:2,000 scale and a diagnosis of the degree of hazard; the location of the zones to be avoided; the recommendations for building and earthworks; and the necessary protective works for achieving an acceptable risk. All these studies have been published in the official journal of the Principality. In the areas where the detailed studies have been performed by either the administration or private promoters, TR for any specific development will simply require the inclusion of the measures recommended in the detailed study.

The completion of the TRs are specially challenging for large landslides because they require, (i) the evaluation of both the reactivation capability of the landslide and the appraisal of rate of the expected movement. In that respect, whether slow moving landslides are able to produce sudden and catastrophic reactivation is a key issue; (ii) the evaluation how changes produced by the construction of earthworks, facilities and buildings may cause on both the overall and local stability of the landslide; (iii) the analysis of the feasibility of the countermeasures for avoiding or mitigating the risk; (iv) the assessment of the effectiveness of the implementation of a warning system and subsequent evacuation protocol. The common issue of these objectives is to find out whether it is possible to live with landslides.
RISK MITIGATION MEASURES: ACTIONS UNDERTAKEN

As it has been already mentioned, in the Catalonia region, the first priority in selecting measures has been directed to land use planning either at county scale or at a local scale. However, just recently, these measures have been promoted systematically for the whole territory. In few specific cases, the protection or landslide stabilisation measures have been considered cost-effective, particularly those related to the main infrastructures (roads and railways). This is the case of the Pont de Bar landslide that was reactivated during November 1982 event and forced the abandonment of this small village. In this landslide both stabilization works and protective measures against the erosion of the Segre river, with a cost exceeding 15 million euros, were carried out (Rodríguez-Ortiz et al. 1988).

Figure 12: General view of the Pont de Bar landslide and the stabilization works. The wall to prevent from river erosion is observable at the centre-left. The abandoned Pont de Bar village appears at the bottom right corner.

The recovery from the catastrophic events of 1982 and 1987 in the Principality of Andorra required intensive engineering works. Shortly after the 1982 floods, the channelization of the Gran Valira river between Les Escaldes and Santa Coloma started with a cost of more than 30 million euros. The rock slide of la Massana in 1987 raised again the concern on the landslides and their consequences on the communities living in the valleys. Several measures were undertaken, such as the construction of a protection gallery in the road at La Massana with a cost of 2.5 million euros.

In January 1997, a rockfall hit a building in the Santa Coloma, a neighbourhood of the capital Andorra la Vella, causing an injury. Rockfall events had also occurred in December 1983 and January 1994 which struck at buildings. The 1997 event forced the Andorran administration to undertake several initiatives. The most important one was the Rockfall Risk Management Master Plan (RFMP) of the Solà d’Andorra which was completed in May 1998 (Copons et al. 2004). This Plan established, for the first time in the Principality, the restriction to the development in the most threatened sectors as it was published in the official registrar of the Principality in July 2000 (Figure 13).
The RFMP defined an upper boundary line above which building is forbidden. The line was published in the official registrar of the Principality in 1998, and since then, it has been used by the Andorra administration for authorization of new developments. When the development line was defined, some of the existing buildings were already within the exclusion area. For all the cases, the RFMP considered the design of defences against rock falls (Copons et al. 2000). Soon after a trajectographic study was finished, the construction of protective works started. The budget for these works was 4.5 million euro (Escalé 2001) and nowadays they are almost completed. Similar actions were undertaken to protect some dangerous stretches of the main roads of the Principality. The performance so far of the protective works has revealed highly effective. Since the fences are operational, several rockfall events have occurred that have been intercepted without further consequences.

Figure 13: Sketch of development zoning in the Andorra la Vella – Santa Coloma area with both the proposed development restriction and the protection measures: 1 - Protection embankments and fences; 2 – Buildings; 3 – Plots non-developable; 4 – Developable plots with protective structures required; 5 – developable plots without restrictions (from Copons et al. 2004).

Figure 14: Construction of protective embankment and rockfall fences in the Santa Coloma area (Principality of Andorra)
The RFMP has been complemented with a Surveillance Plan which was established in 1998. This Plan has several goals (Amigó et al. 2001): (a) inventorying of rock falls occurred in the valley side; (b) validating and updating the trajectographic models used to design the protective structures (path followed, height of bounces, among other parameters); and (c) predicting the potential for large rockfall events (exceeding thousands of m$^3$). It is expected that, in case of development of a large rock mass failure, premonitory signs, such as the increase of small rockfall events or the opening of new fractures, might appear.

**LANDSLIDE RISK PREPAREDNESS**

Landslide preparedness involves both administrative offices and individuals knowing about the hazard potential where they live and taking steps to reduce the potential from the natural hazards. The Catalonian region has implemented a Master Plan for Civil Protection (PROCICAT) with the goal to provide the organizational framework to all the stakeholders. This plan describes the procedures allowing the integrated operation of all the public services and authorities in case of crises. PROCICAT is conceived as a Multirisk Plan.

The Andorra government have developed measures to minimize the effects of the landslides and to protect lives. As it has been mentioned, after the large rock slide of La Massana, a tunnel was constructed to protect the road from future slope failures. This event also evidenced the need of alternative routes to avoid the isolation of the valley as occurred in 1987. Two new mountain roads were constructed connecting the Valira del Nord Valley with that of Valira d’Orient, through mountain passes. However, because of the winter snow, only one of the roads is operational all over the year. In 2005, the construction of a new tunnel started to connect the two main valleys in Andorra (Valira del Nord and Valira d’Orient) that it is expected to be operational in 2008.

The Andorran authorities are also involved in the installation and operation of early warning system. The Geohazard Plan foresees the possibility of development on large landslides evaluated as having moderate hazard category, provided that a detailed analysis has been carried out and that stabilization measures, building requirements, and monitoring systems are implemented. Some dormant or slow moving earthflows, such as that of Canillo, have been considered as having a moderate hazard level. This landslide is moving to an average rate of less than one cm per year and the nature of the slide materials and its geometry suggests that a sudden reactivation of the whole movement is not likely to occur. This is proved by the fact that houses and huts, some of them are more than two centuries old, show only minor cracks and slight structural deformations, while a romanesque-style church of the XI century is still standing. During this span of time, several intense rainfall events have occurred without records of significant reactivation of the landslide. Because of all these observations, it has been considered that living on such landslides might be feasible. However, building authorization will be given only if a reasonable safety factor is obtained in detailed stability analyses of these sites and countermeasures (i.e. protection against river undermining of the landslide toe), restriction of earthworks (i.e. limitation of height of excavation within the landslide body), establishment of a real-time monitoring network and evacuation plans have been adopted. In 2007, works for implementing a monitoring network composed of extensometric wires and piezometers have started. This monitoring network is expected to be fully operational in 2008.
FINAL REMARKS AND CHALLENGES ON LANSLIDE RISK MANAGEMENT

The experiences of both administrations show that landslide inventories and maps at appropriate scales are the minimum requirement for the adoption of the appropriate landslide risk mitigation measures. Furthermore, these maps must be official documents otherwise they might not be taken into account in the land development plans.

The experience of the Andorra Principality has shown that the awareness of the landslide hazard has risen with the public audiences, the building codes and the control works. The success in the adoption of some protective measures is mostly due to the Andorran administration that have taken care of the cost for the construction of the protective works and of the detailed studies foreseen in the Geohazard Plan. This decision has speeded up the processes of developing the threatened areas and delivering of the building permits. On the other hand, the performance so far of the Rockfall Hazard Mitigation Plan has revealed to be highly effective. The most important achievement is that the stakeholders have changed their perception of risk.

Despite the improvements achieved in risk mitigation, there are still several uncertainties and challenges. The quantitative assessment of the landslide risk requires large amount of information, particularly, good quality geological data and of previous landslide activity (particularly, the frequency-magnitude relations), which are difficult to obtain. This is especially true in the case of assessing the probability of occurrence of rare but highly potentially damaging events. Many disasters in the past have demonstrated that, in some cases, complete protection from the landslide threat cannot be achieved. In the Principality of Andorra there exist several locations that may be potentially affected by large landslides that exceed the design event. This is the case of the Solà d’Andorra where the future occurrence of large rockfall events should not be disregarded. In that respect, the Rockfall Surveillance Plan of the rock slope implemented at Solà d’Andorra is a tool to help in the prediction and forecasting of large rockfall events or rock avalanches (exceeding thousands of m³).

Anyway, further research is needed on the assessment of the occurrence of large events. The estimation of the probability of the large landslides based on the occurrence of past events might be too pessimistic. In the Eastern Pyrenees, these events are rare phenomena that occurred under conditions that no longer exist. Most of the large landslides took place in post-glacial times. After the glacier retreat, large amounts of till deposits were swept away in the form of debris flows and avalanches while some over-steepened rock walls made of weak rock collapsed. Dating of many of these large landslides has shown that they occurred preferably just after the glacier retreat (Furdada & Vilaplana 1988; Moya et al. 1997; Turu & Planas 2005) while few evidences have been found of large landslides triggered in historical times. The assessment of the probability of the triggering of future large landslides is the crucial issue. Land use management policies based on the occurrence of rare events having high destructive capability would restrict the development in the main valley without any alternative of relocation.

Finally, the development of slow-moving landslides, such as the one of Canillo, raises the question of the possibility of sudden reactivations (surges). Even though several evidences suggest that this landslide have not experienced global and catastrophic reactivations, the knowledge of the behavior of these types of movements needs to be improved.
REFERENCES


**ACKNOWLEDGEMENTS**

The support of the Andorra Government and that of the EU funded Mountain Risks project (Contract MRTN-CT-2006-035798) is fully acknowledged.
THE ALERTA RIO SYSTEM

Ricardo N. d’Orsi
Geo-Rio Foundation
Rio de Janeiro City Government

Abstract: The Alerta Rio System is the warning system against landslides and severe weather of the City of Rio de Janeiro launched in 1997, the System has been gradually upgraded and introduced to Rio’s inhabitants and civil defense so that at present it is indispensable as a crisis management during events of heavy and prolonged rains. This article presents the System history and some of its most important operational and technical features.

THE SYSTEM CREATION
In June 1991 upon the completion of the Map of Landslide Risks in the Municipality of Rio de Janeiro, the initial sensation of the GEO-RIO specialists who had produced the document was a mixture of pride, relief and of having fulfilled their duty. After all, it involved the first complete mapping of the municipality on a detailed scale (1:25,000), incorporating computer technology (the overlaying of thematic maps, including geology, clinometry, soil use, etc), thirty years of profound knowledge of landslides in Rio de Janeiro, systematic field surveys and the full time dedication of what was still known then as the Geomechanical Research Division.

After the initial impact, a look at the full size map, which is around 2.5 m long and 1 m high, causes a certain uneasiness: around 400 red stains (i.e. areas with a high susceptibility to landmass movements) are spread around the summits of hills and upland areas in the municipality indicating (with undeniable scientific calmness) an alarming situation of risk for many tens of thousands of people.

During this time, the belief that the reduction of risks in Rio de Janeiro could not be based solely on stabilization works (whether preventive or corrective) and on reforestation, as had been happening since the creation of the Geotechnical Institute in 1966, grew increasingly strong. The significant number of people threatened (and the very evident tendency for the rapid and continuous growth of this number) means that to reduce the risk in the municipality there is a need for, in addition to traditional structural interventions (construction work, removals in cases of emergency, reforestation and housing programs), actions of an operational nature, with a wide scope and which can be rapidly put into practice, have to be implemented in the city’s routine. It became evident that there was a need for a system capable of monitoring the approximation and the development of geological and rainfall conditions capable of provoking in a short space of time a large number of landmass movements (especially soil and rock slides) on hillsides in the municipality. This system needed to be capable of informing in real-time and automatically the amount of rain in the entire city and to ‘warn’ whenever critical rainfall indices (capable of triggering landsliding) were reached. Then, the warning needed to be immediately passed on to the general population and to the Municipal Civil Defence, allowing the possibility of the relocation of people to secure places while high risks conditions continued and the optimization of services provided to the population in cases of geotechnical accidents on hillsides during storms. In this way, the idea for a special alert system for Rio was born: the Alerta Rio System.
ALERTA RIO INITIAL PHASE

Five years passed between the initial conception of the system and its effective inauguration. During this time, rainfall-landslide correlation studies and technical visits (to public bodies responsible for environmental monitoring and meteorological equipment companies) interspersed with the development and implementation of a pilot project: the SIGRA (Radio Transmitted Geotechnical Information System) project. SIGRA lasted for 24 months during which time the main technical information necessary to design the Alerta Rio System was obtained. In October 1996, the first telemetric stations began to transmit their reports to the system control room (the ‘Central Station’), installed in the premises of the GEO-RIO Foundation, and by December of that year the entire telemetric network (30 stations) were fully operational. The initial design of the Alerta Rio System included the following characteristics:

1. Telemetric network with 30 automatic raingauge stations, self-sufficient in energetic terms (they possess a solar panel/battery system) and for data transmission (via radio). (Figures 1 to 3)
2. Rainfall records are sent to the Central Station at regular 15-minute intervals, in an uninterrupted manner and with a 0.2 mm precision level.
3. Rainfall records are automatically analysed immediately and in an uninterrupted manner using a specially developed computer program to compare the hourly rainfall intensities accumulated in 24- and 96-hour periods with the critical rainfall indices defined by the GEO-RIO team responsible for the System. The computer program includes a sub-routine with the function of generating automatic ‘warning’ to the team whenever critical indices are reached. The warning is given by the automatic sending of faxes containing rainfall information from all the telemetric raingauges to pre-defined telephone lines (the office of the president of GEO-RIO and the residences of team members).

Figure 1: Location of raingauges in the Alerta Rio System network
The necessary provision of meteorological information during storms (expected duration and intensity of the storm) was provided by meteorologists from INFRAERO (commercial branch of Brazilian Air Force) through a commercial contract. It is worth noting that at that time INFAERO was the only institution in Rio de Janeiro city with a team of meteorologists working on a 24 hour basis.

The criteria initially adopted for the emission of Landslide Bulletin on hillsides in the city (Figure 4) were as follows:

- CRL (Critical Rainfall Level) reached in at least three telemetric raingauges;
- Prediction of heavy rain in coming hours and
- Reports of small incidents on hillsides.

Figure 2: Overview of a raingauge

Figure 3: Overview of a raingauge
Figure 4: Model of Landslide Bulletin used by the Alerta Rio System between 1996 and 1999, faxed to radio and TV stations during the crisis. People are instructed to avoid landslide risk areas and roads and remain in safe places until the bulletin’s cancellation.

This basic configuration of the System lasted from 1996 to 1999. During this period, much was learned about slope behaviour in relation to rain. Rainfall-landslide correlation studies started to be detailed and computer programs were developed to handle the rainfall database and installed in the Central Station computer system. Each new significant rain event gradually increased the number of consultations made over the internet of Alerta Rio’s rainfall records by different individuals and institutions. Free consultation (data is made available without the need for passwords or registration, etc) and the rapid and uninterrupted updating of records (every 15 minutes, in other words practically in real-time), were important aspects in disseminating this public service, which has been implemented in the City.

FROM LANDSLIDE TO SEVERE WEATHER WARNING SYSTEM
At the beginning of 2000, by now having been widely distributed in the media and having become well known internally within the city government, the Mayor requested that Alerta Rio also provides Intense Rain Warning Bulletins.

This new attribute required profound modifications to the general structure of the system: the service provided by Alerta Rio now had to adopt in addition to its principal function (rainfall monitoring and the issuing of alerts of the high probability of landslides on hills) the mission of the ‘meteorological surveillance’ of the municipality of Rio de Janeiro.

With this new responsibility, the system had to detect the approximation of storms before they actually reached the municipality, and to notify the Carioca population and municipal institutions, within as short a period as possible, of the high possibility of the occurrence of severe storms during the coming hours. For this, in addition to the existing structure, a dedicated team (available 24 hours per day) of meteorological specialists and meteorological
radar were required, with the latter having to be capable of identifying the approximation of rain in a radius of at least 250km around the municipality.

New studies, consultations and price surveys (for the acquisition of equipment) and the provision of funds (maintenance and labour) were carried out by technical staff from the GEO-RIO Foundation to structure that new stage of the system and to prepare the necessary Call for Bids to contract the new services. Furthermore, an information exchange agreement was signed between the Rio de Janeiro city government (through the GEO-RIO Foundation) and the Aeronautics Ministry (through the Directorate of Electronics and Flight Protection (DEPV) to guarantee access to meteorological radar images from Pico do Couto (in Petrópolis -RJ).

Finally on 25 April 2000, the new phase of the Alerta Rio System commenced operations, with the official commencement of a new maintenance and operation system for the system. In this new contract, in addition to the maintenance of the telemetric network, which has to present a monthly operability rate of 99%, and the ‘Central Station’ (equipment and programs), meteorologists monitor, working day and night shifts without interruptions, the images generated by the meteorological radar in Pico do Couto and from a large amount of other meteorological information (satellite images, results of numerical forecasting models, etc) available at internet sites. The new contract also allowed two meteorological stations to be included in the telemetric network that, in addition to rainfall information, included wind, temperature, atmospheric pressure and air humidity records.

The adaptation of the Alerta Rio System to deal with the new attributions did not just include the acquisition of new equipment, programs and technical staff. Equally as important was the implementation of the Intense Rain Monitoring Plan (Plano de Monitoramento de Chuvas Intensas - PMCI), which in the final analysis decides the criteria for the issuing (in the event of storms forecast for the next few hours) of Intense Rain Warning Bulletins. Using simple technology, the PMCI consolidates the experience of system operators in the interpretation of all the information that reaches the control room aimed at ‘nowcasting’ related to heavy rain. Like the other products generated by the System, the nowcasting is also updated in real-time on the internet. Chart 1 summarises the characteristics of the different stages in the PMCI, Figure 5 shows the System control room while Figure 6 presents an example of how the Alerta Rio System transmits information referring to rain conditions in the very near future (nowcasting) over the internet.

Figure 5: Alerta Rio control room in the middle of June 2007
Figure 6: (Simulated) example of the indication on the Alerta Rio web site (www.rio.rj.gov.br/alertario) of regions in the city with the different forecasts for rain in the coming hours. In this case the west and south zones in the city are at the ‘Attention’ stage, while the north and central zones are at the ‘Surveillance’ stage and the Barra/Jacarepagua zones are at the ‘Alert’ stage.
Table 1: Intense rain monitoring plan

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
<th>PCI² (%)</th>
<th>Operational Procedures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surveillance</td>
<td>Forecast for coming hours of absence of rain or light to moderate rain without the possibility of intensification</td>
<td>0 – 10 (for the next 6 hours)</td>
<td>Monitoring, organization, analysis and updating of information that arrives at or is generated by the Central Station.</td>
</tr>
<tr>
<td>Attention</td>
<td>Last minute occurrence of moderate rain with the possibility of intensification in the coming hours or Forecast of moderate to heavy rain in coming hours</td>
<td>10 – 40 (for the next 8 hours) or 40 – 70 (for the period covered by the next 3 to 6 hours)</td>
<td>Communication via radio to the Action Group² about the commencement of the Attention stage, followed by complementary information (probable duration of the event, most threatened region of the city, etc.). Monitoring, organization, analysis and updating of information that arrives at or is generated by the Central Station.</td>
</tr>
<tr>
<td>Alert</td>
<td>Last minute occurrence of moderate to heavy rain, with the possibility of intensification in coming hours or Forecast of strong to very heavy rain in coming hours</td>
<td>≥ 70 (for the period covered by the next 2 to 3 hours)</td>
<td>Communication via radio to the Action Group² about the commencement of the Alert stage, followed by complementary information. Issuing of Intense Rain Warning Bulletin by fax sent to radio and television stations. Communication via telephone to television station with the highest audience. Monitoring, organization, analysis and updating of information that arrives at or is generated by the Central Station.</td>
</tr>
<tr>
<td>Maximum Alert</td>
<td>Occurrence of strong to very heavy rain continually in the previous two hours and forecast of strong to very heavy rain in coming hours</td>
<td>≥ 75 (for the period covered by the next 2 to 3 hours)</td>
<td>Communication via radio to the Action Group² about the commencement of the Maximum Alert stage, followed by complementary information. Monitoring, organization, analysis and updating of information that arrives at or is generated by the Central Station. Issuing (via fax) of Alert Cancellation Bulletin and its communication via radio to the Action Group. Return to the Surveillance Stage.</td>
</tr>
</tbody>
</table>

Notes:
1. CI = Intense Rain (precipitation ≥ 30 mm in one hour).
2. PCI= PROBABILITY OF INTENSE RAIN (in each of the four macro-basins that drain the municipality).
3. Action Group: Members of public municipal institutions (GEO-RIO, Civil Defence, Rio-Âguas Foudation, Conservation, Traffic, etc.) that directly work during the occurrence of storms in order to minimise the negative consequences of these climatic phenomena.
CONCLUSIONS
The Alerta Rio System can be considered to be a successful program. In October 2006, the System commemorated ten years in operation during which time constant improvements were pursued and implemented, both in technological updating (equipments and software) and in daily operations (criteria, monitoring modes and alarms for severe weather situations), both in the dissemination and use of its products by the population, civil society and public institutions in the city of Rio de Janeiro. Gradually Alerta Rio has become part of the daily life in the city, helping amongst other things the work of Civil Defence with its precise and updated information and the development of climatic studies in the municipality’s teaching and research centres.

Nowadays, the continuity of the System’s operations is no longer threatened by the cyclical administrative changes that occur in the city every four years, because the new administrators have already recognised the importance of this public service that costs around one Brazilian cent per inhabitant per month.

After forty years of existence of the GEO-RIO Foundation, the histories of the System and the Foundation have begun to merge. Having been since its very beginning the responsibility of that prestigious technical organisation, Alerta Rio has always been assisted (technically and economically) by geotechnical specialists from GEO-RIO. Like its creator and mentor, the System has in a few short years become part of the ‘technical heritage’ of Brazilians and has made its name as a national reference in its area. The System was designed, created and has managed (and continued) to improve the lives of Cariocas.
REPORT ON LANDSLIDE IMPACTS AND PRACTICES IN SWITZERLAND: NEED FOR NEW RISK ASSESSMENT STRATEGIES

M. Jaboyedoff
Institute of Geomatics and Analysis of Risk
AMPHIPOLE, University of Lausanne, Switzerland

Ch. Bonnard
Independent Landslide Expert, Lausanne, Switzerland

Abstract: Switzerland is a mountainous country with high density population. Owing to climate changes and human activities, the impacts of landslides are increasing. Recent events are described and inspected in terms of risk strategy. It appears that Switzerland is at a turning point in its landslide risk strategy, because the changing physical and social conditions have induced new situations: the events are more intense, global warming changes the landscape of the Swiss mountains and more infrastructure increases the number of exposed elements. The discussion focuses on the preoccupation of a rich population, which makes Switzerland a laboratory for the evolution of risk management.

INTRODUCTION
This report presents some aspects of landslide activity since 2005 with some less recent examples. The choice of the different events and case studies is guided by the different issues linked to landslide risk management and strategies that exist in this country. It must also be said that Switzerland is a multicultural country, formed by 26 independent cantons, which means that the French speaking authors will mainly present case studies from the western part of Switzerland.

Switzerland has an average population density of 183 inhabitants/km$^2$, rising to over 305 inhabitants/km$^2$ when excluding the mountainous regions, and reaching 594 inhabitants/km$^2$ in some urbanized cantons (OFS 2005). More than six percent of the Swiss territory is affected by landslides (Noverraz & Bonnard 1990; Lateltin 1997). The expenses for natural hazard in Switzerland reach 0.6% of GDP or 2 billion USD, which represents 4.7% of the Federal state, 3.7% of the Cantonal (i.e. regional authority) and 5.7% of the municipal budgets. Among these expenses, 750 million USD are dedicated to risk prevention, and 300 million USD are used for emergency actions.

Landslide risk management is an ancient preoccupation in Switzerland (Heim 1932), because of the historically high population density with respect to high landslide hazard, which was increased by deforestation especially in the 18th and 19th centuries. A landmark paper by Einstein (1988), presented the first landslide risk review, in which the DUTI (1985) program was presented as an example. This research programme was the first synthesis attempt for landslide hazard mapping in Switzerland. Nowadays, climate change and economic pressure (in particular tourism) make the situation in Switzerland very difficult to manage.

If the global warming was around 0.6°C for the 20th century, it reaches up to 1.6°C in western Switzerland, 1.3°C in Swiss-German part and 1°C in the south Swiss Alpine region (OcCC 2007). In the past, the Alps followed a more rapid warming than other regions (IPCC 2001).
This shows that the Alps, like other cold regions, are more sensitive to climate change. Many events linked to permafrost thawing or glacier retreat and melting are now documented, making the Alps more hazardous than other regions.

This is underlined by the fact that, during the summer 2006, the Swiss population became quite aware of the problems of rock mass movement in the Alps due to the dramatic one-month closure of the European highway crossing Switzerland by the Gotthard Tunnel (Liniger & Bieri 2006). Before that, the 2003 heat wave (PROCLIM 2005) over Europe induced a very high rockfall activity, affecting among others, the famous Matterhorn (Gruber et al. 2004). In Switzerland, at least 975 deaths are attributed to the heat wave, leading to an increased mortality rate by 7% (PROCLIM 2005). The increase in extreme events (OcCC 2007), especially intense precipitation events, makes Switzerland more hazardous to landslides and particularly to shallow landslides. Furthermore the economic development of mountainous areas increases the exposed infrastructures and holiday houses.

Switzerland should be finalizing its program of landslide hazard mapping, as funding will be cut if this program is not completed in 2011. This program follows the forest law in force (LFO 1991), clearly including the protection of population and transportation routes. The following sections present some significant case studies (Figure 1) providing a particular focus on the history and perspective of risk management in Switzerland.

Figure 1: Location of the different cited case studies

HISTORY
Switzerland suffered several large, more or less famous landslide events in its history, amongst them the landslides in Elm (1881; 155 deaths) and Goldau (1806; 457 deaths) which were the foundations for landslide sciences through the work of Heim (1932) (Eisbacher & Clague 1984). In 2006, the bicentenary disaster of Goldau was officially commemorated with an interesting scientific meeting, which underlines that in Switzerland, catastrophes can be considered as the cement of the Swiss nation, because of the mutual assistance between Swiss
regions (Pfister 2002). It is obvious that the landscape of Switzerland is shaped by numerous landslides and particularly by landslides having occurred since the last glaciation of the Alpine valleys (Bonnard, in press). Several huge landslides occurred after or during the last maximum glacial retreat. The Sierre rockslide (1 km$^3$) probably propagated partly on ice (Heim 1932; Burri 1997). The famous Flims sturzstrom (~9500 B.P.; 12 km$^3$) still presents a great interest for the international scientific community (Heim 1932; Pollet et Schneider 2004; Deplazes et al. 2007; von Poschinger 2005).

As in many European countries, the problem of the overexploitation of forests in Switzerland increased landslides, floods and snow avalanches hazards within the Alpine territory. As a consequence, the Swiss authorities already promulgated a forest protection law in 1876, which can be considered as one of the first laws integrating natural risk management and risk reduction strategies (FOEN 2001).

**THE RECENT HISTORY**

Landslide hazard mapping recommendations were published in 1997 (Lateltin 1997), and have initiated a normative landslide mapping program in Switzerland. Since that time, the so called “danger maps” survey is active. However, even if the National Platform for Natural Hazards (www.planat.ch) enacts recommendations (Figure 2) for risk strategies, but there is no law dedicated to natural hazards risks. On the contrary, an ordinance (RS-455.1) adopted in 1991 concerns major accidents related to dangerous material. This was the consequence of a major chemical catastrophe in 1986 through a fire in a chemical industry of Basel (Sandoz), polluting the Rhine river and killing most of the life downstream as far as the Netherlands. This ordinance states that for chemical industries and the transport of dangerous chemical substances, a risk analysis must be performed for which the ALARP principle is applied (FOEN 2007). This strategy is not yet transposed to natural hazards, despite various proposals formulated in European research projects, like Imiriland (Bonnard et al. 2004).

![Figure 2: Risk wheel or sequence of risk management viewed by the Swiss federal offices (FOEN 2007b)](image)

The basis for the landslide risk hazard and management strategy in Switzerland was caused by Falli Hölli landslide in 1994. The mass of around 40 million m$^3$ moved 200 m in 4 months, with a maximum velocity of 6 m/day, destroying a tourist settlement of 41 chalets. The home owners were indemnified for a total of 15 million USD (Lateltin & Bonnard 1995). Three
years after this event, the Swiss federal authorities published the Swiss recommendations for mass landslides and their consideration in land planning (Lateltin 1997).

The series of catastrophic events that may be related to climate change initiated by the mid-October 2000 storm event that killed 16 people and caused about 550 million USD damage. The most catastrophic event in terms of human lives occurred in Gondo, killing 14 people in one unique event caused by a relatively small landslide of 10,000 m$^3$, which broke a concrete wall built for rockfall protection. Another important event in mid-October 2000 was a debris flow of approx. 400,000 m$^3$ flowing down to the village of Fully, situated in the Rhone Valley stopping at the limits of the village (FOEN 2002). The altitude difference between the source area and the deposit is at least 1000 m. The debris flow source is located within fluvial-glacial deposits and/or scree slopes. This catastrophe seems to have a human origin. An overflow of a small lake at the top of the penstock caused its failure and induced the debris flow. In 2006, a localized storm over the southwestern part of the Alps also produced a large debris flow. This event derailed a train, but luckily, no causalities were resulted. The disturbing fact is that the debris followed a path with a 2000 m change in altitude.

![Figure 3](image_url)

**Figure 3:** (a) Debris flow in Brienz (from FOEN 2007b). (b) Landslide within a thin moraine cover overlying gneissic basement induced by the heavy rainfall of August 2005 closing the road to Morcles village on August 23, 2005 (Photo courtesy of E. Bardou, IGAR-UNIL). (c) Slide in Entlebuch valley (from Bezzola & Hegg 2007).

In 1991, in Randa, a rockfall of 29 million m$^3$ fell down in two episodes (22 million m$^3$ on April 18th and 7 million m$^3$ on May 9th) from the highest cliff of the area, which is the largest rockslide of Switzerland in the 20th century. It cut the road and the regional train. It occurred because of unfavourable fault and joints, rock fatigue and intense snowmelt that led to over
pressure within the gneissic rock mass (Sartori et al. 2003). It did not produce a rock avalanche, despite the large volume, because the mass fell down in several episodes of small volumes. The rockfall created a dam in the river, which induced a situation of high risk, because of potential failure. A channel was dug successfully, however heavy rainfalls induced two floods in the village of Randa. The direct cost of the works and damage was approximately 65 million USD.

In 1996, two important rockfall events occurred in Sandalp, the first with a volume of 0.47 million m$^3$ on January 24$^{th}$ and the second of 1.75 million m$^3$ on March 3$^{rd}$ (Keusen 1998). The deposits obstructed a valley and posed a problem similar to the damming of the Mattervispa after the Randa event (1991; 30 million m$^3$). Sandalp is the largest rockfall in Switzerland since Randa.

During the autumn in 2002, a rock mass of approximately 100,000 m$^3$ of broken gneiss began to move with unusual velocities above the village de St Niklaus, just a few kilometres north of Randa. In October 2002, the movement accelerated and important precipitations increased the destabilization of the cliff. The rockfall occurred on 22$^{nd}$ November, while the population had been evacuated and an embankment was under construction. The risk management during the crisis was remarkably led by a local geological company that had already evacuated the exposed inhabitants three times previously.

The 2003 heat wave over Europe induced numerous rockfalls in high mountain areas. During the heat wave of summer 2003, the Matterhorn, one of the symbols of Switzerland, suffered from a 1000 m$^3$ rockfall caused by thawing of permafrost (Noetzli et al. 2004). Since that event, the impacts of climate change seem to be more and more perceptible. Climate change causes either more intense rainfall events (Fallot 2000) that seem to be induced by apparently new climatic conditions, which are lowering the return period of heavy precipitation, or an increase of average temperatures.
On November 29, 2003, along the main road joining Switzerland to Italy through the Gd. St Bernard tunnel, a rockfall of 600 m$^3$ pierced a snow avalanche protection gallery killing one person. The road was reopened under restrictions 2 days later, and completely reopened 3 weeks later. Since that period, the cliff where the rockfall occurred is monitored using real-time monitoring network, GUARDA VAL, that includes extensometer, temperature, precipitation data, etc. (Rouiller et al. 2004). Eight active instabilities are monitored continuously in the canton of Valais.

Finally, since the mid 18th century, the La Frasse landslide movements have been monitored and studied (Noverraz & Bonnard 1988; Tacher et al. 2005). The movements reached up to several metres per week during past crises. After some 150 years of studies aiming at understanding this landslide and local actions trying to mitigate its effects, a drainage gallery is now being excavated below the slip surface. Boreholes from inside the gallery will reach the moving mass and the landslide will be drained by gravity, reducing its crises by some 95%.

**SOME RECENT EVENTS**

In the present section, we discuss some of the events occurred since 2005 that are significant in terms of process types and issues linked to risk management.

**2005 Events in Central Switzerland**

From August 18 to 23, 2005, particular meteorological conditions brought hot humid air from the Mediterranean Sea towards the north. When that air met fresh air, up to 300 mm precipitation fell in 5 days (up to 190 mm in 24 hours) in the northern part of the Swiss Alps (Bezzola & Hegg 2007). The central part of Europe was also affected by this event. In Switzerland, 5 people were killed by landslides: 2 by a debris-flow, 1 by a rock fall and 2 by a mudflow. The total amount of damage reached about 2.1 billion USD among which 1.4 billion USD were insured, which is very high (Swissre 2006). 900 communities were concerned, and more than 5000 landslides were reported in central and western Switzerland (Bezzola & Hegg 2007) (Figure 3). We note that in the area of Brienz (River Glyssibach), the perimeter of the debris flow that killed 2 people suffered a similar event in 1846 as well as several others during the 19th century (River Trachtbach). This shows that the increase of population density in the Swiss Alps, as well as higher property values, lead to increasing vulnerability, being the main cause of disasters, in spite of well documented high risk areas. This becomes a major issue for the management of natural risk.

Through examining the spatial distribution of landslides and precipitation, the correlation is clearly strong (Figures 4 and 5). The return period for 48 hour (21 to 22 August) precipitations is 77 years (Bezzola & Hegg 2007), indicating that such an event can be quite frequent, taking climate change into account. The density of landslides per square kilometres was computed using a 5 km search radius around the grid points. The precipitation for 72 hours was resampled on the same grid. The plot of the results of both grids shows that, globally, no landslide occurred if the precipitation was below 70 mm/3 days, except in a few cases (Figure 6). Furthermore, the maximum density does not pass the value given by the following equation:

\[
\text{Density} = 8.335 \times \ln (\text{precipitation over 72h}) - 35.4 \ \text{[events/km}^2\text{]} \quad [1]
\]

It does not reach more than 7 landslides per km$^2$, and the average number is not above 1
This shows that the corresponding hazard level can be calculated from such computations and can be used for risk analysis.

Figure 5: Density of landslides per square kilometer and simplified precipitation patterns for 3 days (72 hours) in mm from August 20 to 22, 2005 (After data from MeteoSwiss 2006; Bezzola & Hegg 2007)

Figure 6: Computation of grid data displaying the density of landslide per square kilometer and precipitation values for 3 days (72 hours) in mm from August 20 to 22, 2007 (After data from MeteoSwiss 2006; Bezzola & Hegg 2007). The red line is the upper limit and the blue line is the moving average over 101 points.
Expropriation of Houses at Vallamand

The northwestern region of Switzerland belongs to the geological units called “Molasses plateau”, mainly formed by sandstones and marls (Becker 1973). These rocks are the “basement” of the most populated area in Switzerland. In May 2006, after rockfalls and slides, the authorities decided to evacuate and destroy 15 houses located at the toe of a cliff near the lake of Morat (Figure 7). This area implies one of the largest expropriation in residential area that occurred in Switzerland. The expropriation cost was around 12.5 million USD. This zone had been known for its repeated hazard events since 1991.

Figure 7: Vallamand landslides and houses that had been evacuated (modified from http://www.geoplanet.vd.ch)

Rockfalls along the Gotthard Road

A major rockfall event (5,000 m$^3$), in terms of impact on society, occurred on Wednesday May 31, 2006. Eleven blocks of around 10 m$^3$ reached the highway crossing the Alps at the Tunnel of the Gotthard (Liniger & Bieri 2006). They hit several vehicles and one of the boulders crushed a car, killing two German tourists (Figure 8). The highway was reopened on Friday June 2nd for one hour, before new rockfalls occurred, which did not reach the highway. The road was again closed for one month. Six million vehicles used this road in 2005 among which 1 million are trucks. The bypassing itinerary by the San Bernardino Pass suffered an excess of 260,000 vehicles in June 2006 (www.swissinfo.org). The month of June was used to clean a 5,000 m$^3$ unstable cliff located within the slope above the highway by blasting. The blocks produced by the blasting stopped above the highway.

The instability was located at about 1,400 m a.s.l., and was therefore not affected by thawing of permafrost. Three blocks stopped on the parking place and just above in March 2004, indicating that the instability was already activated or reactivated. Since this event, studies are now performed in great detail in this area.
Eiger Collapse

In June 2006, the eastern flank of the Eiger was affected by a fast movement of the rock mass that suffered the progressive collapse of about 4 million m$^3$. This activity began on June 11 by a rockfall of approx. 1,000 m$^3$, opening a steep back crack (Figures 9 and 10). High-resolution Lidar monitoring indicated that the instability was divided in two main blocks, the front moving forward at 15 cm/day and rear block moving down by 70 cm/day (Keusen et al. 2007; Oppikofer et al. in review). The Lidar monitoring allowed the forecasting of the July 13, 400,000 m$^3$ rockfall. The unstable mass is till now collapsing; its behavior is now closer to a granular material than a classical structural instability as it was at the beginning. The cause of this collapse is certainly the position of the rock apron that was weakened by the glacier activity (pressure, retreats and advances). The currently retreating glacier, still lying below the collapse, is probably the trigger of this instability.

Figure 9: View of the 4 million m$^3$ instability on June 20 (left) and September 22 (right), 2006. The cliff is 250 m high (Photo courtesy of H.-R. Keusen, GEOTEST)
This event was a real attraction during summer 2006 in the Swiss Alps: tourists and media were constantly present observing the erosion of the mountain, especially because of the earlier Gotthard event. Fortunately, no direct risks were present. The only problem was a potential creation of a lake behind the deposits, but fortunately the underlying ice at the toe of the cliff allows the transit of water.

**Durnand**

On Tuesday July 26, 2006 in the evening, a local train-track Martingy-Orsières was cut by a debris flow of around 25,000 – 30,000 m$^3$ (Figure 11). The rails were broken or moved by more than 400 m. The cause of this accident is a storm of an extreme violence, with precipitations of more than 50 mm/h.

Figure 10: Lidar 3D image of the rock face showing the displacement between 11 and 12 July 2006 (rear block: 70 cm/day and front block 10-15 cm/day). The circled zone was moving at around 1 m/day, which allowed the forecasting of the July 13 collapse (right picture) (modified after Oppikofer et al. in review).

Figure 11: (A) View of the former location of the historical bridge and of the debris on the right side, taking note of the important section and the height of the debris flow at this place. (B) Train that was pushed by the debris-flow, 3 days after the event.
The train running at 60 km/h could fortunately stop; none of its 34 occupants were injured. During the night, 3,000 inhabitants of the town of Martigny downslope were evacuated. According to an interpretation, a blockage occurred at level of the old bridge, as the deposits around on both sides of the bridge attested it. The material accumulated forcing the bridge to fail.

The material of the debris flow originates from morainic material and rock debris produced by active rockfalls covering the dead ice that flows down the slope. This material supplies the top of the stream at an elevation of about 2300 m a.s.l. where the debris flow initiated. The material travelled for approximately 5 km down to the railroad (Figure 12). Along the path, the volume increased by erosion, leaching completely a certain parts of the stream bed. The upper part of the valley is mapped as potential permafrost (FOEN 2007). Furthermore it is also clear that ice lies below the scree. July 2006 was the hottest since 1864 in the northern part of Swiss Alps (Fallot pers. com.; Meteoswiss 2006). As a consequence, under such conditions, a storm in July was certainly greatly increased by the ice melting. This is highly suspected by the fact that the triggering zone is located within the moraine and/or scree-debris near hard rock outcrops, which represents a drastic change in permeability.

Figure 12: (A) Location and path of the debris flow and approximate location of the triggering zones (1). (B) View of the triggering zones. (C) View of the upper part of the scree-debris accumulation (Photo courtesy of J.-D. Rouiller CREALP)
Dents-du-Midi Rock Instabilities
On October 30, 2006, a 1 million m$^3$ rockfall collapsed from the Dents-du-Midi (approx. 2900 m a.s.l.), the mountain located far behind the picturesque Château de Chillon (Figure 13). This event was caused by an unusually hot autumn. The deposit did not reach any settlements, but only pasture areas. High mountain rock avalanches are still a risk for human activity especially in tourist area.

Rockfall at Les Diablerets
In May 2006, after heavy rainfall in the municipality of Les Diablerets, a farmer worker was killed by a rockfall of several blocks totalling approximately 20 m$^3$. The blocks’ material was microconglomerate and it fell along a dip slope formed by the strata. The block source was located in a steep slope close to the top of the mountain located above a high mountain pasture (Figure 14). The blocks did not reach the inhabited area, occupied from May to October, but hit the worker who was reinstalling barbed wire fence after the winter.

Figure 13: View of the rockfall coming from the summits and ending not so far from settlement (Photo courtesy of J.-D. Rouiller CREALP)

Figure 14:  (A) Rebound locations of four block trajectories measured using differential GPS. (B) 3D trajectories using Rotomap © software and a 5 m DEM. (C) View of the larger block which followed the left trajectory on (A) (modified after Maillard et al. 2007)
This small event was not widely highlighted in the local media, nevertheless this event underlines two interesting points: (1) 3D modelling of the rockfall indicates that using either 5 m DEM or 2 m DEM, we were not able to simulate the trajectory of blocks. These problems have already been pointed out by Agliardi & Crosta (2003). Furthermore, it shows that the gyroscopic effect is still not very well taken into account within present codes (Dorren, pers. com.). (2) This event induced the local authorities to initiate the study of hazard mapping, which are still not completed in the canton de Vaud.

Mud Flow in Les Diablerets
After the same heavy rainfall event that caused the rock fall in Les Diablerets, a mudflow in the same region cut a small mountain road located at 1750 m a.s.l. close to tree line altitude. The area contains several small shallow landslide scars within moraines and colluviums. Most of them are related to track cuts. The 2007 mudflow was approximately 500 m$^3$, and affected grey and brownish moraine (Figure 15). It appears that the moraine is weathered that the pore pressure can be affected by such a modification. The main point is that a water pipe was located in the upper part of the scar. The triggering factor was heavy rainfall, which was not exceptional (probably a return period of less than 3 years).

For more than 30 years, the cowsheds in Switzerland have been equipped with running water. This occurred mainly as transportation has been facilitated on mountainsides; the Swiss federal government began favouring the creation of mountain roads in the early 1970s. Nowadays subsidies to maintain mountain territory and infrastructures have been drastically
reduced. As a consequence, most of the infrastructures are ageing, and are subject to more dysfunctions.

**Debris Slide of Pont Bourquin (Les Diablerets)**

A debris slide occurred on July 5, 2007, near the settlement of Les Diablerets on a main road going to the Pillon passing down to Gstaad. The material was part of a larger shallow landslide of around 40,000 m$^3$, which belong to larger landslide affecting the entire slope. The movement initiated in around 2004 (Figures 16 and 17). It was apparently a shallow slide of around 80 cm deep in the upper part of the slope that increases its volume by retrogressive erosion at the change of inclination (Figure 18). This erosion produced approximately 3000 to 6000 m$^3$ of debris that were accumulated down slope. They heavy rainfall of July 2007 mobilized this material, which reached and cut the road (Figure 17).

![Figure 16: Cross-section along the major slide indicating the debris](image1)

![Figure 17: Front of the debris slide that was partly cleared from the road](image2)
The landslide affects mostly schist overlying flysch, the schist producing the debris. Below, gypsum is found, that is probably the cause of landslides in the whole slope, mainly driven by gypsum dissolution. This event is significant of the recent slides in Switzerland, because:

1. It is a reactivation of erosion product and located within a larger system of slow moving landslides affecting the whole slope.
2. An erosion area developed within schist and has extended since the 1990s.
3. The forest is becoming old, without maintenance, and has grown on former pasture, indicated by farms ruins.
4. Some chalets are now renovated for tourism and running water is often installed.

This example signifies that old forests do not necessarily provide stabilization and that a simple perturbation in a semi-abandoned area can lead to erosion and increased landslide activity, especially because these areas are no longer of economic interest, mitigation is quite difficult at reasonable costs. These areas of erosion have developed in numerous locations that are characteristic of the Swiss geology. It will become a preoccupation mainly because of the lack of survey.

Figure 18: Evolution of the erosion surface over a period of 9 years. In 2004, the scar is visible in the upper grassy part (data: SWISSIMAGE © 2004 swisstopo DV012716).

LESSONS FROM THESE EVENTS
The above mentioned events indicate different issues linked to natural risk management. As we have seen, the major problems are the land planning and the increasing activity in the Alps that led to conflicts between hazardous zones and building or infrastructure areas. The Swiss population is forced to live with risks, nevertheless the population asks for more safety. Thus, in some cases, as in the Gotthard example, the risk was too high to be tolerable, especially for the European populations transiting through this highway. On the one hand, the authorities are more prepared to perform expropriation, because, in high risk areas, it is not possible to accept risk, such as in Vallamand. But, on the other hand, the incredible price of the property market are currently so high in the Alpine tourist areas that it is difficult to apply very strict rules for land planning. This makes it difficult to define a risk strategy. However, in this context the media are playing an increasing role by putting forward some facts that are
not necessarily important, as it was the case for the Eiger collapse. This is also mixed with climate change issues, which are linked without any serious analysis of landslides by journalists. Nevertheless the link with climate change is unquestionable; the processes that affect the mountain summits can reach the valleys because of the temperature increase and cycles of freezing and thawing.

Shallow landslides are more sensitive to extreme meteorological events and they produce the greatest threat to human activity, i.e. ageing infrastructures, surface water concentrations and slope profiling. They have created new issues, as they are numerous and potentially very destructive. It can also be postulated that the decreasing number of freezing days and the weathering of moraines can also be a worsening factor for the mechanical behavior of slopes. In addition, shallow landslides can affect areas without significant economical interest because, in these areas, erosion can progressively increase as it is the case for Les Diablerets. Such erosion areas need to be taken into account in a longer term perspective because they can increase rapidly in surface area and create huge problems in the future, as before the law on forest. This implies to develop an improved understanding of landslides and modeling capacities. However, the latter are not sufficient because of their limitations.

Until recently, the direction of the civil protection was controlled by the military. It has recently become temporarily a separate entity from army, now it depends again on the federal department of defense and safety. The wheel of Figure 1 shows clearly that the strategy is centered on catastrophe, and is a reactive strategy. Nowadays, the potential changes in slope behavior and management (unbelievable price paid by rich tourists for chalets in some ski resorts) make reasonable land-use planning difficult, if risk is not integrated in risk management.

This implies that territories now have to be inspected everywhere, especially above important routes or settlements. This strategy has been initiated by the federal agencies since the accident of the Gotthard. This event has also underlined that current "danger maps" used for land-use planning in Switzerland are somehow unsatisfactory to manage risk in the Alps, because they are prepared in development areas and not in high mountain zones. Thus data collected during hazard studies must include: frequencies, intensities, but also scenarios etc. Again, this shows the need for new concepts, and also integrated risk management. The main problem of such a strategy lies in the prioritization which can be performed using cost-benefit analysis; this is possible only if hazard assessment has been performed, which implies detailed studies and if vulnerability is assessed, which is not the case. However, limited budgets imply that other tools must be used to prioritize actions, which can be done for instance using hazard impact matrix (Krause et al. 1995).

The long closure of the Gotthard demonstrates the inconsistency of the decision between reopening the road for one hour and a one month closure. The risk management must integrate an adaptation to situations and not a reaction to events as Figure 1 shows.

Because Switzerland is a wealthy country, it possesses a very high level of insured objects as the 2005 events have shown. It also underlines that global risk management works in Switzerland is efficient: hazard mapping will be probably completed by the end of 2011, because subventions will be cut at that date. In some Cantons like Vaud (state of 3200 km²), this constraint has created a dynamic that will lead to an ambitious hazard mapping program, including risk management. However, this process creates an apparent feeling of safety. The risk linked with the economic impact of budget reductions in several domains such as forestry,
will induce an increase in hazards because of the lack of the management of “non-economic” territories. Nevertheless the general high safety level makes Switzerland a laboratory for the evolution of risk management in a context of climate change and property market. There is still an apparent contradiction between an implicitly risk-based management (events with return period higher than 300 years are not taken into account in land planning) and the desire of the population to be exposed to no risk at all.

Another unexpected effect of the three rockfalls: Matterhorn, Eiger and Dents-du-Midi is an impact on the symbols of Switzerland, and in fact a direct destruction of what Switzerland represents. This is probably a sign that something has changed in this country. The risks are no longer only economic problems, but also symbolic.

CONCLUSIONS
Switzerland being a wealthy country, it has suffered several recent catastrophes that have killed dozens of persons, but until now, these incidents were within the ALARP zone, as no laws have been dedicated to natural hazards risk management until now. This limit was reached for the Gotthard case. Secondly, even if the year 2005 was less affected than others in terms of number of events, one single intense meteorological event made it the most expensive year in terms of costs due to natural events in Switzerland (Bezzola & Hegg 2007). Thus the hazard impacts on Swiss society are changing. The needs are different, but are we prepared even if we are investing a lot of money? It is not sure. Another issue is the attitude of Swiss authorities, who during the heat wave of 2003, said that Switzerland did not face the same problems as our neighbors France and Germany. We now know that 975 people died for this reason - this is comparable to the level of the other countries. This reveals some problems within the risk management hierarchy in Switzerland.

Switzerland was a pioneer in natural hazard prevention and monitoring concepts, as indicated by the Kilchenstock management proposed by Heim (1932) and underlined by the first forest law (1876). Nowadays, the economical pressures and hyper regionalism of Switzerland leads progressively to some blockages in the risk management application strategy. Furthermore, the strategy at the federal level is dictated by Swiss Germans who do not necessarily share the same risk culture and organization goal with the rest of the country. As we are speaking of risk, a certain culture of risk exists, but is this strategy compatible with a multicultural country?

Risk management in Switzerland is facing societal changes. Being rich, Switzerland has the possibility to respond to this evolution, which cannot be assessed in poorer countries because there is little time for such questions. With respect to former mitigation policies there is clearly a trend towards an adaptation of the acceptable risk depending on the importance of exposed elements. Switzerland is a laboratory to create a new risk management strategy owing to the new climatic, economic and anthropogenic conditions; this will be our challenge for the future, but we do not know now if we will succeed.

REFERENCES


Lateltin, O. (1997). Prise en compte des dangers dus aux mouvements de terrain dans le
81-86.
http://www.admin.ch/ch/f/rs/c921_0.html.
http://www.meteoschweiz.admin.ch/web/fr/services/apercu_produit/bulletins_meteoro
logiques/bulletins_meteorologiques4.html.
Maillard, B., Veuthey, C., Sanchez, E., Zufferey, T., Fauquex, N., Rossier, A., Bruchez, J., and
Noverraz, F., and Bonnard, Ch. (1990). “Mapping methodology of landslides and rockfalls in
Switzerland.” Proc. VIth Int. Conf. and Field Workshop on Landslides, Milan, 43-53.
OcCC (2007). Climate Change and Switzerland in 2050. Impacts on Environment, Society
Climate and Global Change, 28 p.
transport of the Holocene Flims sturzstrom (Swiss Alps).” Earth Planet. Sci. Lett.,
221, 433-448.
télésurveillance adapté aux régions de montagne.” 57ème Conférence canadienne de
géotechnique - Québec, 25-30.
Randa rockfalls (Canton of Valais, Switzerland). Natural Hazards and Earth System
Sciences, 3, 423-433.
large landslide with respect to hydrogeological and geomechanical parameter
heterogeneity.” Landslides, 2, 3-14.
33-47.

ACKNOWLEDGEMENTS
We thank the County geologist of Valais, who provided us pictures and information, E.
Bardou, T. Oppikofer and A. Pedrazzini for the figures, A. Loye for discussions and K.
Sudmeier for English language improvement.
LANDSLIDE RISK MANAGEMENT:  
COUNTRY REPORT FOR NORWAY

Suzanne Lacasse and Farrokh Nadim  
International Centre for Geohazards  
Norwegian Geotechnical Institute, Oslo, Norway

Abstract: The paper provides an overview over the threat of landslides and other natural hazards and the present (November 2007) situation for dealing with hazard and risk of slides in Norway. Landslides caused by heavy rainfall, flood, erosion, and human activities represent the most common threats on land. Near-shore and offshore, different geological processes, earthquakes and anthropogenic activities, for instance in connection with petroleum exploration and production, can trigger slides and large mass flows. The paper devotes most attention to quick clays slides and rock slides.

INTRODUCTION

“Geohazards”, i.e. natural hazards that are driven by geological features and processes, pose severe threats to humans, property and the natural and built environment. During 2005, geohazards accounted for about 100,000 deaths worldwide, of which 84% were due to October’s South Asia earthquake. In that year, natural disasters affected 161 million people and cost around US$160 billion – over double the decade’s annual average. Hurricane Katrina accounted for three quarters of this cost. During the period from 1996 to 2005, natural disasters caused nearly one million lives lost, or double the figure for the previous decade, affecting 2.5 billion people across the globe (World Disaster Report 2006). Over the decade, 51 people died per natural disaster event in countries of high human development, compared to 573 deaths per event in countries with low human development (Centre for Research on the Epidemiology of Disasters, Belgium). When the trend of fatalities due to natural hazards is studied over the last 100 years, it appears that the increase in the known number of deaths is due to the increase in the exposed population in this time scale and the increased dissemination of the information, and not to an increase in the frequency and/or severity of natural hazards.

NATURAL HAZARDS IN NORWAY

The main natural hazards in Norway are landslides, snow avalanches, floods and, to a lesser extent, earthquakes (Solheim et al. 2005a). Statistically, 10 large slides can be expected to occur in Norway in the next 50-100 years, each with possibly 20-100 associated deaths. The number of lives lost due to all types of slides over the past 150 years exceeds 2000.

Rock Slides

As the last glaciers receded, about 11,000 years ago, large portions of east and mid-Norway were left covered with clay deposits, while in other areas (western and northern Norway) high mountains rose and deep valleys were eroded. Many of the mountain sides and leached clay deposits were unstable, and, based on the landslide scars and moraine residues observed today, a large number of slides took place. The sliding activity also took place offshore in the North Sea and Norwegian Sea. Today, the profession knows that a number of large rock slides and clay slides occurred between 5,000 and 10,000 years ago. The sliding activity in
Norway has continued since the last ice period, but less frequently than immediately after the retreat of the glaciers (Lied 2008). The sliding activity is expected to continue in the future.

Rockfalls and rockslides are among the most critical geohazards in Norway, mainly because of their tsunamigenic potential. The largest known rock slide in Norway is Tjelleskredet in Langfjorden in Romsdal in 1756. The 15 million m$^3$ slide triggered a tsunami with a height up to 50 m, causing 32 fatalities (Jørstad 1956). The natural disasters causing the largest number of deaths in Norway in the 20$^{th}$ century involved large rock slides into fjords (narrow bodies of water) having generated a tsunami: Loen in 1905 (350,000 m$^3$, 40 m tsunami, 61 fatalities), 1936 (1,000,000 m$^3$, 74 m tsunami, 73 fatalities) and Tafjord in 1934 shown in Figure 1 (1,500,000 m$^3$, 62 m tsunami, 41 fatalities) (Helland 1905, 1911; Helland & Steen 1895; Holmsen 1936; Grimstad 2005). In Loen, a rockslide of 1,000,000 m$^3$ also occurred in 1950, but caused no fatalities.

Quick Clay Landslides

About 5000 km$^2$ of Norway is covered by soft marine clay deposits. Nearly 20% of this area consists of highly sensitive or quick clay. Landslides in quick clay represent a common and important threat, especially in Norway and Sweden (and parts of Canada). Landslides in quick clay are frequently triggered without warning and turn into a flowing liquid in a matter of minutes, and they can progressively involve very large volumes of soil. Statistically, at intervals of about 4 years, large quick clay slides with moving clay masses of several million cubic metres occur in Norway (Aas 1979, 1981).

The largest quick clay slide in Norway in the 20$^{th}$ century occurred at Rissa near Trondheim in 1978 (Figure 2, left, Gregersen 1981), covering an area of 330,000 m$^2$. Near 6 million m$^3$ of clay moved out at high velocity. The largest historical quick clay slide in Norway occurred on 20 May 1893 in Verdal, north of Trondheim, where 55 million m$^3$ of clay ran out, and 116 fatalities were recorded. This is the largest known natural catastrophe in Norway in historical times.

A typical quick clay slide consists of a minor initial slide followed by a progressive failure process developing very rapidly in all directions from the first slide. The Rissa quick clay slide started with the failure of a small fill by a lakeside. The initial slide involved only 200 m$^3$ of sediments. It grew to 6 million m$^3$ in a few hours through retrogressive sliding.
Avalanches
Snow avalanches represent the most frequent type of sliding in Norway. Most of the slide-related casualties in Norway are due to avalanches (Figure 3). Several thousands of snow slopes fail each winter in the mountainous and fjord areas of Norway (Hestnes 1985; Kristensen 1998; Lied & Kristensen 2003; Lied & Sandersen 1989). The majority of avalanches takes place in deserted areas and cause little damage. However, many lives have been lost (1,500 fatalities since 1836), and long interruptions in communications arteries have occurred because of avalanches. The road and railroad system have also suffered greatly from avalanches, but vehicles have rarely been taken by them. Dwellings are often destroyed by avalanches. In 1979, 11 persons died and over 100 farm residences and barns and 160 construction camps and leisure cabins were damaged by avalanches in Norway. The cost of the material damages was about 200 MNOK (25 M€) in that one year (Domaas & Lied 1980).

Figure 2: Quick clay slides in Norway: the Rissa slide, 1978 (left) and the Trøgstad slide, 1967 -1 million m³, 4 fatalities (right)

Figure 3: Avalanches in Norway: 1968 Riise avalanche (right)

Other Natural Hazards
The seismicity of Norway (Figure 4a) and adjacent areas is moderate and, even though it is the highest of north-western Europe, it is still lower than in many other ‘stable’ continental intra-plate regions (Bungum et al. 2005). Seismicity rates during the 20th century suggest that the region experiences, on average, one magnitude (M) 5 earthquake every 10 years and one M 7 earthquake every 1100 years. An overview of the data behind these numbers is given in Figure 4b, including a frequency–magnitude distribution that is reasonably stable except for the largest events where the time period covered is too short. Specific earthquakes in the 20th century included two earthquakes with magnitude greater than 5 offshore western Norway
(1988, 1989), one M 5.4–5.6 earthquake in the Oslo fjord region in 1904, in addition to a few nearby offshore earthquakes of about the same magnitude. The largest known earthquake in historical times in Norway is a M 5.8 earthquake that occurred in 1819 in the Rana region.

![Evidence of a large prehistoric earthquake on the Stuoragurra (Masi) fault in North Norway (part of the Mierujavr’i–Sværholt fault zone)](image)

The average annual cost of flood damage in Norway is about 200M NOK. However, there are large variations from year to year. After a major flood in south eastern Norway in 1995, a governmental commission gave several recommendations to reduce future flood damage. The cost of the damages caused by the 1995 flood amounted to about 1.8 billion NOK (200M €). One of the recommendations of the commission on flood protection measures in Norway was to produce flood inundation maps for the areas susceptible to the largest damage. This is an ongoing activity of the Norwegian Water Resources and Energy Directorate (NVE) (www.nve.no).

**Underwater Slides**

The exploitation of offshore petroleum resources, development of oil and gas pipeline corridors, fishing habitat protection, and protection of coastal communities, have contributed to a growing interest in Norway for underwater slides, in particular seafloor mass movements and their consequences. The development of the Ormen Lange field, which is the second largest gas field on the Norwegian Continental Shelf, contributed greatly to the understanding of underwater slides. The field is in the Norwegian Sea in water depths 800 to 1,100 m, approximately 120 km from the coastline, and within the scar of the prehistoric Storegga slide (Figure 5). The Storegga slide, which took place 8,200 years ago, is one of the world’s largest known submarine slides with an estimated slide volume in excess of 3,000 km$^3$ and run-out of 300 km (e.g. Solheim et al. 2005a, 2005b). Evidence of a major tsunami generated by the Storegga slide has been found along the coasts of Norway, Scotland and the Faeroe Islands. Considering the enormity of the Storegga slide and the potentially catastrophic consequences of a similar event today, it was essential to clarify and quantify the risks associated with submarine slides in the area to obtain approval for field development from the authorities (Nadim et al. 2005; Bryn et al. 2005). The numerous studies carried out in the Ormen Lange offshore geohazards study were summarised in a special volume of Marine and Petroleum Geology journal in 2005.
HAZARD ZONATION IN NORWAY
Norway, like many other countries, has seen increased vulnerability to landslides and increased awareness of the need for mapping, due to industrial and recreational development over the entire country, infrastructure development, the consequences of interruption in the communication arteries and increase in population. A few major disasters in the past 30 years also helped “convince” the authorities to take preventive measures. However, compared to...
100 years ago, the risk may be higher, even if a large number of areas in Norway have already been subjected to mapping and mitigation studies. To increase safety and reduce hazard and risk, and to assist with emergency preparedness, a priority mapping is underway in Norway for landslides in clays, rock slides and snow avalanches. Susceptibility mapping has been done continuously in Norway since the late 1970s. The hazard and risk maps are especially useful for prevention and the planning of new dwellings, schools, recreation areas, etc. The entire network of communication corridors and military and humanitarian (Red Cross) exercises have need for such maps (Karlsrud et al. 1984; Gregersen 1989).

LEGISLATION AND ADMINISTRATIVE PROCEDURE
The Building and Planning Act in Norway has been under development since 1924 and the act was put into force for the whole country in 1966. The last revision was done in 1987. The Building Act is used when a detailed hazard plan is made with corresponding detailed maps. The on-going hazard mapping on survey maps at 1:50,000 scale has been operative since 1979, and up until now, approximately 110 maps are completed. Still 100 maps need to be prepared and an estimated 15 more years will be required to complete the work. So far the maps have no legal liability, but are used to help land-use planning in the communities. The building council of the counties has to follow the rules stated in the Act. Advice on hazard zones and protective measures are given by geo-consultants. For avalanche-endangered houses that are older than 1966, the National Fund for Natural Disaster Assistance can fund rebuilding with protective measures or moving the houses. In 1980, a new Act stated that all objects with Fire Insurance need to also take out Natural Hazard Insurance. Damages caused by natural hazards will normally be compensated in full, unless the owner has shown gross negligence. Insurance companies do not initiate hazard assessment or recommend safety measures. They may, on the other hand, increase the insurance premium or refuse permission to rebuild (Hestnes & Lied 1980).

Today, ROS-analyses (risk and vulnerability analyses) are run before regulation plans and building requests can be approved by the county authorities. At the county level, the project proponent needs to establish whether the area is susceptible to landsliding. For the regulation plan to be approved, the proponent needs to determine if there is a hazard and the potential consequences. In the building plans, the proponent needs to document safety or prepare mitigation measures. The estimation of hazard for natural events is connected to the Norwegian Planning and Building Law. According to the technical regulations in the law, three classes of avalanche and slide frequencies should be taken into account (Table 1).

Table 1: Safety class in technical regulations in the Norwegian planning and building law

<table>
<thead>
<tr>
<th>Safety class</th>
<th>Maximum nominal frequency (per yr)</th>
<th>Return period (yrs)</th>
<th>Type of construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$10^{-2}$</td>
<td>100</td>
<td>Garages, smaller storage rooms of one floor, boat houses</td>
</tr>
<tr>
<td>2</td>
<td>$10^{-3}$</td>
<td>1000</td>
<td>Dwelling houses up to two floors, operational buildings in agriculture</td>
</tr>
<tr>
<td>3</td>
<td>$&lt;10^{-3}$</td>
<td>&gt;1000</td>
<td>Hospital, schools, public halls, etc.</td>
</tr>
</tbody>
</table>
There is also a fourth class, where the consequences are so important that the buildings cannot be placed in a “hazard zone”. How one determines a “no-hazard” zone is however not defined. The building regulation states that rebuilding after fires or other kinds of reparation may be done for Class 2, when the nominal yearly frequency is less than $3 \times 10^{-3}$, i.e. a return period of 333 years or more. By using the word “nominal” as opposed to “real”, one admits that the exact calculation of avalanche run-out distance for the given frequencies is not possible, and that the use of subjective judgment is necessary. The rules were first established for the mapping of snow avalanche hazard.

**RISK ZONATION FOR QUICK CLAY**

As part of work for the Norwegian Water Resources and Energy Directorate (NVE), Gregersen (2001) developed a simple method to classify and map the risk posed by potential quick clay slides. Potential slide areas are given “engineering scores” based on an evaluation of the geotechnical parameters, local conditions, persons or properties exposed and engineering judgement. Hazard classes are described as low, medium and high. Consequence classes are discussed as not severe, severe and highly severe. The resultant risk, based on engineering evaluation and experience, is divided in five risk classes (Lacasse et al. 2004).

**Hazard Classes**

The hazard level depends on topography, geological and geotechnical conditions, and changes at the site. The evaluation of the hazard is done with the help of Table 2. The weight given to each hazard in Table 2 (or later, to consequence in Table 3) describes its importance relative to the stability of the slope. The hazard classes are:

- **Low**: Favourable topography and soil conditions; extensive site investigations; no erosion; no earlier sliding; no planned changes, or changes will improve stability.
- **Medium**: Less favourable topography and soil conditions; limited site investigations; active erosion; important earlier sliding in area; planned changes give little or no improvement of stability.
- **High**: Unfavourable topography and soil characteristics; limited site investigations; active erosion; extensive earlier sliding in area; planned changes will reduce stability.

The zones with weighted score between 0 and 17 (up to 33% of maximum score) are mapped as “low hazard” and have low probability of failure by sliding. The zones with weighted score between 18 and 25 (up to 50% of maximum score) are mapped as “medium hazard” and have a higher, though not critical, probability of failure. The zones with weighted score between 26 and 51 are mapped as “high hazard” and have a relatively high probability of failure.

**Consequence Classes**

Consequences are commonly evaluated in terms of human life safety, environmental, financial and social effects. The evaluation of the consequences is done with the help of Table 3, with consequence classes:

- **Not severe**: No or small danger for loss of human life, costly damage or consequences.
- **Severe**: Danger for loss of life or property or important economical or social loss
- **Highly severe**: High exposure of human life loss or large economical or social loss.

The zones with weighted score between 0 and 6 (13% of maximum score) are mapped as “not
severe”. In these zones, there would be very few or no permanent residents. The zones with weighted score between 7 and 22 (up to 50% of maximum score) are mapped as “severe”. The zones with weighted score between 23 and 45 are mapped as “highly severe”; they would hold a large number of persons, either as residents or as persons on the premises temporarily.

Table 2: Evaluation of hazard for slides in quick clay in Norway

<table>
<thead>
<tr>
<th>Hazard</th>
<th>Weight</th>
<th>Score for hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>- TOPOGRAPHY</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Earlier Sliding</td>
<td>1</td>
<td>Frequent, Some, Few, None</td>
</tr>
<tr>
<td>Height of slope, H (^1)</td>
<td>2</td>
<td>&gt;30 m, 20-30 m, 15-20 m, &lt;15 m</td>
</tr>
<tr>
<td>- GEOTECHNICAL CHARACTERISTICS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overconsolidation ratio (OCR)</td>
<td>2</td>
<td>1.0-1.2, 1.2-1.5, 1.5-2.0, &gt;2.0</td>
</tr>
<tr>
<td>Pore pressures (^2)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- In excess (kPa)</td>
<td>3</td>
<td>&gt; +30, 10-30, 0-10, Hydrostatic</td>
</tr>
<tr>
<td>- Under pressure (kPa)</td>
<td>-3</td>
<td>&gt;-50, -(20-50), -(20-0), Hydrostatic</td>
</tr>
<tr>
<td>Thickness of quick clay layer (^3)</td>
<td>2</td>
<td>&gt;H/2, H/2-H/4, &lt;H/4, Thin layer</td>
</tr>
<tr>
<td>Sensitivity, S(_t)</td>
<td>1</td>
<td>&gt;100, 30-100, 20-30, &lt;20</td>
</tr>
<tr>
<td>- NEW CONDITIONS</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Erosion (^4)</td>
<td>3</td>
<td>Active/sliding, Some, Little, None</td>
</tr>
<tr>
<td>Human activity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Worsening effect</td>
<td>3</td>
<td>Important, Some, Little, None</td>
</tr>
<tr>
<td>- Improving effect</td>
<td>-3</td>
<td>Important, Some, Little, None</td>
</tr>
<tr>
<td>TOTAL SCORE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum weighted score</td>
<td>51</td>
<td>34, 16, 0</td>
</tr>
<tr>
<td>% of max. weighted score</td>
<td>100%</td>
<td>67%, 33%, 0%</td>
</tr>
</tbody>
</table>

Notes:
1) For the quick clays in the study, inclination was identical for all slopes (1:3), and slope inclination was not included as a variable. In a general study, slope inclination should be added in the list of hazards.
2) Relative to hydrostatic pore pressure
3) In general, the extent and location of the quick clay are also important.
4) Erosion at the bottom of a slope reduces stability.

**Risk Classes**
The risk score to classify the mapped zones into a risk class is obtained from:

\[
\text{Risk} = \text{Hazard} \times \text{Consequence} \\
R_{WS} = H_{WS(\%)} \times C_{WS(\%)} \tag{1}
\]

where \( R_{WS} \) = Weighted score for risk mapping  
\( H_{WS(\%)} \) = Hazard weighted score in \% 
\( C_{WS(\%)} \) = Consequence weighted score in \%
Table 4 gives the risk scores for the five risk classes used for quick clay slides in Norway. Figure 6 shows a risk mapping of the area Modum in Norway.

Table 3: Evaluation of consequence for slides in quick clay in Norway

<table>
<thead>
<tr>
<th>Possible damage</th>
<th>Weight</th>
<th>Score for consequence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3</td>
<td>2</td>
</tr>
<tr>
<td>HUMAN LIFE AND HEALTH</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Number of dwellings</td>
<td>4</td>
<td>&gt; 5 Closely spaced</td>
</tr>
<tr>
<td>Persons, industry building</td>
<td>3</td>
<td>&gt; 50</td>
</tr>
<tr>
<td>INFRASTRUCTURE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roads (traffic density)</td>
<td>2</td>
<td>High</td>
</tr>
<tr>
<td>Railways (importance)</td>
<td>2</td>
<td>Main</td>
</tr>
<tr>
<td>Power lines</td>
<td>1</td>
<td>Main</td>
</tr>
<tr>
<td>PROPERTY</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Buildings, value</td>
<td>1</td>
<td>High</td>
</tr>
<tr>
<td>Consequence of flooding</td>
<td>2</td>
<td>Critical</td>
</tr>
<tr>
<td>TOTAL SCORE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum weighted score</td>
<td>45</td>
<td>30</td>
</tr>
<tr>
<td>% of max. weighted score</td>
<td>100%</td>
<td>67%</td>
</tr>
</tbody>
</table>

Notes:
i) Permanent residents, in both sliding area and within run-out distance.
ii) Normally no one on premises, but building(s) have historical or cultural value
iii) Slides may cause water blockage or even dam overflow; flooding may cause new slides; there should be time for evacuation; damage depends on a complex interaction of several factors.

Table 4: Risk classes for slides in quick clay in Norway

<table>
<thead>
<tr>
<th>Risk Class</th>
<th>1 (lowest)</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5 (highest)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Risk Weighted Score ($R_{WS}$)</td>
<td>0-160</td>
<td>167-600</td>
<td>628-1900</td>
<td>1906-3200</td>
<td>3200-10,000</td>
</tr>
<tr>
<td>$R_{WS}$ (% of max $R_{WS}$)</td>
<td>0-1.6%</td>
<td>1.6-6%</td>
<td>6.3-19%</td>
<td>19-32%</td>
<td>32-100%</td>
</tr>
</tbody>
</table>

**Decision-making on Remedial Measures**
To make decisions on the need for additional soil investigations, stability analyses or other remedial actions, Table 5 gives recommendations for quick clay areas in Norway.
Table 5: Activity matrix as a function of risk class

<table>
<thead>
<tr>
<th>Activity</th>
<th>Risk class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-2</td>
</tr>
<tr>
<td>Soil investigations</td>
<td>None</td>
</tr>
<tr>
<td>Stability analyses</td>
<td>None</td>
</tr>
<tr>
<td>Remediation</td>
<td>None</td>
</tr>
</tbody>
</table>

1) e.g. erosion protection, stabilizing berm, unloading, soil stabilization, moving of residents

Figure 6: Quick clay risk map for Modum, Norway

**DEBRIS FLOW HAZARD ZONATION**

Mapping of debris flow hazard involves the following main considerations:

- The hydrological regime of the drainage basin
- The availability of sediment sources
- The stability of the river channel on the entrance of the run-out zone

Drainage basins responding quickly on rainfall or snowmelt input are the most dangerous. Lakes or gently inclined wetlands will store water and lead to elongated flow hydrographs, while steep basins with less water storage capacity will have more concentrated hydrographs and higher debris flow activity. The largest sedimentation fans are normally associated with
the last category of drainage basin. Models for estimation of hydrological response can be used to evaluate runoff of different recurrence intervals.

The debris flow activity is the highest in rivers which have abundant steep and unstable side slopes. Geomorphological maps will give valuable information on the extent of sediment sources within the drainage basin. Field inspection of these locations is needed to evaluate the stability and the quantity of potential sediment supply.

The spreading of the material into the fan area depends, to a large degree, on the stability of the river channel. Deep incised channels with stable side-slopes will prevent lateral spreading. In shallow and unstable channels, the possibility for lateral spreading is much higher. Only careful field inspection can give answer to this problem.

In Norway until recently, run-out estimates were mostly based on empirical and topographical models. The run-out can be estimated by the inclination of the straight line connecting the run-out with the point in the track where sedimentation is the predominant process. This point of sedimentation usually coincides with bed inclination of 15° in confined and 20° in unconfined channels. Stones with diameter greater than 10 cm can reach down to 13°, while the sand fraction can go as far as 10° (Figure 7).

Figure 7: Run-out distance of debris flow by topographical factors

The volume of the sliding material is probably the most important factor for the extent of the run-out zone. If several millions of cubic metre is involved, the run-out cannot be evaluated by simple dynamic or topographic models. This is especially important if large rivers are blocked and huge amounts of water are dammed with the possibility of generation of catastrophic flood waves downstream.

EXAMPLE OF EARLY WARNING SYSTEM FOR ROCK SLIDE
Rock falls and rockslides are among the most dangerous natural hazards in Norway, mainly because of their tsunamigenc potential. The three most dramatic natural disasters in Norway in the 20th century were tsunamiis triggered by massive rockslides into fjords or lakes (Loen in 1905 & 1936 and Tafjord in 1934), causing more than 170 fatalities (Bjerrum & Jørstad 1968; Anda & Blikra 1998). As public attention on natural hazards increases, the potential rockslides in the Storfjord region in western Norway have earned renewed focus. A massive rockslide at Åknes could be catastrophic as the rock slide-triggered tsunami is a threat to all the communities around the fjord. The Åknes/Tafjord project was initiated in 2005 by the municipalities, with funding from the Norwegian government, to investigate rockslides, establish monitoring systems and implement a warning system and evacuation plan to prevent
fatalities, should a massive rockslide take place.

Åknes is a rock slope over a fjord arm on the west coast of Norway (Figures 8 and 9). The area is characterised by frequent rockslides, usually with volumes between 0.5 and 5 millions m$^3$. Massive slides have occurred in the region, e.g. the Loen and Tafjord disasters (Figure 1). Bathymetric surveys of the fjord bottom deposits show that numerous and gigantic rockslides have occurred many thousands of years ago. The Åknes/Tafjord project (www.aknes-tafjord.no) includes site investigations, monitoring, and an early warning system for the potentially unstable rock slopes at Åknes in Stranda County and at Hegguratksla in Norddal County. The project also includes a regional susceptibility and hazard analysis for the inner Storfjord region, which includes Tafjord, Norddalsfjord, Sunnyslvsfjord and Geirangerfjord. The potential disaster associated with a rockslide and tsunami involves many parties, with differing opinions and perceptions. As part of the on-going hazard and risk assessment and validation of the early warning system, event trees were prepared by pooling the opinion of engineers, scientists and stakeholders. The objective was to reach consensus on the hazard and risk associated with a massive rockslide at Åknes (Lacasse et al. 2008).

![Figure 8: Location map of the study area in Åknes/Tafjord project](image)

**Observed Displacements**

Experience from Norway and abroad shows that rockslide events are often preceded by warning signs such as increased displacement rate, micro-tremors and local sliding. Accelerating rate of displacement several weeks and even months before a major rockslide event is typical. Slope movements have been detected at Åknes down to 60 m depth (Figure 10). New borehole data suggest movements down to 100 m. Important uncertainties lie in the most likely failure depth and location, and whether the slide will occur as one large 30-60 millions m$^3$ sliding event or a succession of several ‘small’ slide events. Figure 10 presents the Åknes slope and two slide scenarios. Figure 11 shows some of the displacements observed at the upper crack. Water seeps (“springs”) are seen emerging on the downstream slope (Kveldsvik et al. 2008). The displacements in Figure 11 appear to move linearly with time. The total annual displacements vary from less than 2 cm up to about 10 cm.
Instrumentation and Monitoring
The large variations in weather and atmospheric conditions in the fjord and mountain areas pose unusual challenges to the instrumentation. For example, the hazard due to snow avalanche and rock bursts is high in most of the area to be monitored. Solar panels do not provide sufficient electricity, and energy has to be obtained from several sources to ensure a stable and reliable supply. Significant effort is underway to deploy robust instruments and improve data communication during periods of adverse weather. An Emergency Preparedness Centre is located in Stranda. The monitoring data will be integrated into a database that will form the basis for future analyses. Based on the experience with similar projects and the specific needs in Storfjord, the overall monitoring system was equipped with:

Surface Monitoring
- GPS-network with 8 antennas
- total station with 30 prisms
- ground-based radar with 10 reflectors
- 5 extensometers measuring crack opening
- 2 simple lasers measuring opening of the 2 largest cracks
- geophones that measure vibrations

Monitoring in Borehole
- Inclinometers measuring displacements
- piezometers measuring pore pressure
- temperature
- electrical resistivity of water

Figure 9: Approximate map of observed or derived rock slide events in post glacial times in the Storfjord region (Blikra et al. 2002).
Figure 10: Sliding volume scenarios. Surficial area (left) and cross-section (right) (modified from Blikra et al. 2007). Area I: Slide volume 10-15 million m$^3$, displacement=6-10 cm/yr. Area II: Slide volume 25-80 million m$^3$, displacement=2-4 cm/yr

Figure 11: Location of extensometers and displacements from extensometer 1, 2, 3, 4 and 5 at the top scarp at Åknes (Kveldsvik et al. 2006).

**Meteorological Station**
- temperature
- precipitation and snow depth
- wind speed
- ground temperature
- radiation

Light Detection and Ranging (LiDAR) mapping was recently done. Several independent systems were installed to ensure continuous operation at all times, and different communication systems were implemented to ensure continuous contact with the Emergency Preparedness Centre in Stranda. Radar measurements were also made, an example is shown in Figure 12.

**Modelling of Tsunami Following Rock Slide**
Figure 13 shows the contour (white dotted line) of the potentially unstable rock mass at Åknes. The length of the “top scarp” (upper crack) is about 700 m. The years on the figure give the approximate dates of rock slides that have occurred earlier, and the black dotted lines
show the assumed surficial area for each of the slides.

Figure 12: Deformations at Åknes measured with radar: displacements 16 Aug.–9 Sept. 2006 (left) and 10-28 Sept. 2006 (right) (LISA-Radar & NGU) (Personal communication 2007).

Figure 13: Potential Åknes rock slide (white dotted line shows possible surficial extent of rock slide)

The tsunami wave propagation due to an Åknes rock slide was modelled numerically for two rock slide scenarios: slide volume of 8 million m$^3$ and 35 million m$^3$. Run-up values were estimated for 15 locations in the Storfjord region (Eidsvig & Harbitz 2005; Glimsdal & harbitz 2006; Eidsvig et al. 2008). The results of the simulation for three locations are shown in Table 6 below (see Figure 8 for locations). Preliminary results of tsunami modelling suggest an inundation height of up to 35 m at Hellesylt for rockslide volume of 35 million m$^3$ at Åknes. The modelling of the tsunami caused by the rockslide includes several uncertainties. To reduce the uncertainties, physical modelling is presently underway in university laboratories in Oslo and Trondheim (University of Oslo and the Norwegian University of Science and Technology (NTNU) in Trondheim. The model tests are run to improve the
understanding of the initial wave pattern generated by the sliding rock masses. A rock slide as large as 30 million m$^3$ will pose a serious threat to coastal areas of several communities in the Storfjord region, and may have also serious consequences further out along the fjord.

Table 6: Estimated run-up heights for locations in the Storfjord region

<table>
<thead>
<tr>
<th>Location</th>
<th>Beach slope inclination (degrees)</th>
<th>Run-up heights (m), 8 million m$^3$</th>
<th>Run-up heights (m), 35 million m$^3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hellesylt</td>
<td>10˚</td>
<td>8-10</td>
<td>25–35</td>
</tr>
<tr>
<td>Geiranger</td>
<td>6˚</td>
<td>8-15</td>
<td>20-40</td>
</tr>
<tr>
<td>Stranda</td>
<td>30˚</td>
<td>1-3</td>
<td>3-6</td>
</tr>
</tbody>
</table>

Early Warning and Emergency Preparedness
The Åknes/Tafjord early warning and emergency preparedness system was implemented early 2008. As part of this system, the Emergency Preparedness Centre in Stranda is in operation continuously (24 hours, 7 days). Alarm levels and responses are under development. The aim is to establish guidelines for monitoring and alert levels as a function of observed displacement rates on the extensometers, in the case of impending failure. Figure 14 and Table 7 present an example of the alarm and response system. The system is in constant evolution. The evaluation of the alarm status is done on the basis of an integrated interpretation of all measurements available, and their evolution over time (Blikra et al. 2007; Blikra, 2008).

Table 7: Sketch of alarm levels and response indicated in Figure 14

<table>
<thead>
<tr>
<th>Alarm level</th>
<th>Activities and alarms</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1 Normal situation</td>
<td>Minor seasonal variations</td>
<td>EPC staff only Technical maintenance</td>
</tr>
<tr>
<td>Level 2 Awareness</td>
<td>Important seasonal fluctuations for individual and multiple sensors Values&lt;excess thresholds for Level 2</td>
<td>Increase frequency of data review, compare different sensors Call in geotechnical/geological/monitoring expert</td>
</tr>
<tr>
<td>Level 3 Increase awareness</td>
<td>Increased displacement velocity, seen on from several individual sensors Values&lt;excess thresholds for Level 3</td>
<td>Do continuous review, do field survey, geo-expert team at EPC full time Inform police and emergency/preparedness teams in municipalities</td>
</tr>
<tr>
<td>Level 4 High hazard</td>
<td>Accelerating displacement velocity observed on multiple sensors Values&lt;excess thresholds for Level 4</td>
<td>Increase preparedness, continuous data analysis Alert municipalities to stand prepared for evacuation</td>
</tr>
<tr>
<td>Level 5 Critical situation</td>
<td>Continuous displacement acceleration Values&gt;excess thresholds for Level 4</td>
<td>Evacuation</td>
</tr>
</tbody>
</table>

Note: EPC = Emergency Preparedness Centre in Stranda.

LANDSLIDE RISK MANAGEMENT IN NORWAY
The goal for landslide risk management in Norway is to offer to all citizens an ALARP risk level (As Low As Reasonably Practicable risk). There exist today databases, methods and
systems for analysis and qualification/quantification of hazard and consequences, and requirements that need to be satisfied before building in new areas or rebuilding in existing areas can be approved. The classification arrived at through scoring has provided good results so far. Even in some cases (avalanches), prototype testing is done, and early warning systems are being implemented in a few selected areas where the risk is perceived as high.

Norway has however an important challenge in making the many actors involved in landslide risk management work together. The many different geographical areas needing mitigation need to be ranked in order to spend the funding in the most effective way to reduce the risk associated with landslides (also avalanches).

There is on-going work in Norway at the ministries’ level to improve the coordination in Norway for all types of slides in Norway (clay slides, rock slide, debris flows, avalanches and underwater slides). Both organisation and responsibilities will be re-examined, also in light of increased hazard and risk due to climate and demography changes. The responsibility for the coordination was given to the Norwegian Water Resources and Energy Directorate (NVE). The Directorate will have responsibility for preparing guidelines and ensuring efficient operation for the following aspects:

- National strategy for landslide management
- Hazard assessment, preparedness and emergency response under acute situations
- Landslide warning
- Detail mapping of hazard, vulnerability and risk
- Quantifying landslide hazard and risk
- Safety measures, mitigation and early warning systems
- Compensation in case of damages
- Increased awareness, education programs etc on landslides

**CONCLUDING REMARKS**

Compared to many areas of the world, the human losses caused by natural hazards are small in Norway and other Nordic countries. This is mainly due to the low population density in the
exposed areas. However, the economic losses are significant and the number of lives lost could be reduced through proper mitigation measures. The predominant natural hazards in Norway are quick clay slides, snow avalanches and rock slides, especially those with potential of triggering a tsunami. Quick clay slides pose a major threat in Norway and Sweden. Slide-triggered tsunamis represent a threat to parts of western Norway. Snow avalanches represent an important threat in the winter.

There are several components of landslide management in place today (2007) in Norway. However, its organisation could be greatly improved by concerted actions where responsibilities are clearly defined and the actions are steered from one Ministry or responsible directorate. An inter-Ministry commission is working on the organization, responsibility and response and preparedness to landslide events in Norway. The report to Government is due mid-2008.

The results of recent research on effect of climate change and landslide frequency imply that the temporal and spatial distribution of natural hazards in Nordic countries might change significantly in the coming decades because of climate change. Adapting to the new situation would require a proactive approach from the politicians, geoscientists and decision-makers.

REFERENCES
seabed Instability at Ormen Lange.” Ormen Lange - An Integrated Study for Safe Field Development in the Storegga Submarine Slide Area, Solheim et al., Elsevier, 311-318.


APPENDICES
Appendix A – Natural Hazards in Other Nordic Countries (from Nadim et al. 2008)
APPENDIX A
NATURAL HAZARDS IN OTHER NORDIC COUNTRIES (from Nadim et al. 2008)

SWEDEN
The major geohazards in Sweden are landslides, floods and snow avalanche. Most mass movements in Sweden are formed in connection with snowmelt and the thawing of frozen ground, as well as intense or prolonged rainfall, i.e. when the water pressure in the ground is high, or when there are sharp fluctuations in the groundwater level. Large landslides have become increasingly common during the past century, most likely as a result of anthropogenic interventions, e.g. construction activity that undercuts or overloads dangerous slopes/flatlands. The landslides in Sweden mostly occur in relatively gentle slopes made up of glacial silt and clay, mainly in quick clay. Most of the dangerous slopes are bordered by open water (river or lake). About 4% of the land surface consists of clay and silt. Based on known landslide scar frequency (Figure A1), about one per cent of the land area is highly susceptible to spontaneous landslides.

Many of the largest and most costly landslides in Sweden have taken place in the River Götaälv valley and its surroundings between Lake Vänern and Gothenburg. The total volume eroded by landslides and gullies in that valley is calculated to be roughly 500 million m$^3$. The latest large landslide with significant casualties occurred at Tuve in 1977. The slide severed seven electric cables and completely destroyed 65 single-family houses, and caused nine fatalities. The consequences of a landslide are not always proportional to its size. In 1918, a small landslide caused a train with about 300 passengers to crash at full speed into the slide. Fire broke out in the wrecked wagons. Some 40 victims were identified, but the exact number of casualties is not known.

Landslides also represent a major threat to the communication infrastructure in parts of Sweden. For example, as recently on 20 December 2006, part of Highway E6 near Munkedal in Bohuslän in the west of Sweden collapsed in a landslide (Figure A2). The E6 is the main road between Oslo and Gothenburg. The landslide caused major disruption to traffic as the 15,000 vehicles that pass the collapsed section of road every day had to be re-routed for several weeks. The site of the landslide was the most recently opened section of the E6, which has been undergoing rebuilding work. The cause of the landslide is not known, but there is speculation that the construction activities and the unusually heavy rain that has recently hit the area could have triggered a quick clay slide. The landslide also destroyed the railway embankment and cut an electricity line leaving the railway without power.

Almost every year, Sweden is affected by floods resulting in damage. Damage can be limited through prevention planning and effective response operations during flood emergencies. For this purpose, the SRSA (Swedish Rescue Services Agency) compiles and maintains general flood inundation maps (Figure A1). These maps are created as basis for prevention work with the help of a watercourse model for those areas close to watercourses that are at risk of flooding. The maps are for use during the planning of emergency and rescue services work and as a foundation for land use planning by municipalities. They can also be used as basis for hazard, vulnerability and risk analyses.
Figure A1: Most susceptible landslide areas (left) (www.sgu.se) and flood risk map (right) (www.srv.se) in Sweden

Figure A2: The 2006 Munkedal landslide affecting the Oslo-Gothenburg traffic

ICELAND
Iceland is primarily exposed to volcanic and seismic activities and avalanches. Frequent earthquakes and eruptions occur along the boundary between the North-American and
Eurasian plates that runs through Iceland as a series of seismic and volcanic zones (e.g. Einarsson 1991; Sigmundsson 2006). A comprehensive record extending over 1100 years as well as present-day good instrumentation and extensive research have revealed the nature of hazards (Haraldsdottir et al. 2006).

Snow avalanches have claimed over 600 lives throughout Iceland’s history (Bjornsson 1980). In the period between 1901 and 2000, landslides claimed 27 lives and snow avalanches 166 lives (Jóhannesson & Arnalds 2001). Presently, the avalanche risk is the most pronounced in threatened coastal villages and a major avalanche protection plan is being carried out. In recent decades, the largest natural disasters in Iceland were two snow avalanches in NW-Iceland in 1995 that claimed 34 lives.

Landslides, in particular debris flows on slopes in fjord environments, occur regularly and cause considerable damage in Iceland. The slopes are glacially over-steepened, and are typically covered with shallow regoliths comprised of till and colluvium. The main source of debris is the rapidly weathered basaltic cliffs that form the upper parts of the slopes and the inherited glaciogenic material still available on the intermediate benches. Debris flow activity in Iceland has been recorded to occur all year round, but the activity level is higher in late spring, late summer and autumn. There are regional differences in the meteorological factors for debris flow initiation (Decaulne & Saemundsson 2007) such as snowmelt, prolonged and intense rainfall and a combination of both.

FINLAND
The European-wide natural hazard maps developed by the ESPON 1.3.1 Hazards project (www.gtk.fi/projects/espon) clearly convey the impression of Finland being a country with few natural hazards. The dominant geohazards in Finland are avalanches, landslides, storm surges and winter storms (Schmidt-Thomé & Kallio 2006). Landslides occur especially on river shores in southern and western parts of Finland among thick clay areas and in the hilly areas in northern Finland. River floods, which may trigger landslides, are also a prominent natural hazard (Jarva & Virkki 2006).

DENMARK AND GREENLAND
Denmark is a lowland (the highest point about 170 m above sea level), with the peninsula Jutland and a number of islands between the Baltic Sea and the North Sea. Denmark is not seriously affected by geohazards. There is no volcanic activity and earthquake activity is of minor importance. The last time serious damage due to an earthquake recorded was in 1842, but the event did not cause any casualties. There is no documented damage to the coastal areas of Denmark by tsunami either. The main geohazards problem in Denmark is landslides, which occur frequently along the cliffed coasts (Pedersen et al. 1989). The landslides in Denmark can be differentiated into rock falls and mud-dominated landslides (rotational slides). The rock falls are related to chalk cliffs and they occur mainly in late winter to early spring, when groundwater saturation is the highest and the action of freeze and taw than triggers the rock falls.

Mud-landslides are common along the coastal cliffs in Denmark. They develop by progressive back-stepping of crescent-formed “décollement” surfaces. The formation of the landslides is controlled by three factors: 1) Steep sloping surface; 2) High pore-water flow (generally depending on the lithology); and 3) Erosion of the toe of the slide. The mud-
landslides cause constructional problems, but no human casualties. These landslides create a problem in parts of the attractive coastal environments. However, the cost of private property is not regarded high enough compared to the cost of coastal protection, thus no active protection or mitigation is provided by the public services.

Greenland is the largest island in the world with and area of 2,166,000 km$^2$, of which 20% are bedrock. The remaining area is covered by the inland ice reaching a thickness of more than 3 km. Very few geohazards problems have affected the Greenland population, which consists of about 60,000 inhabitants. There is no recent volcanic activity in Greenland. Earthquakes, caused mainly by the displacements in the inland ice, are of minor importance. Landslides and rock falls, however, can cause serious problems.

The landslides in Greenland are influenced by permafrost, glacial ice, high topographic relief, and repeated freezing and thawing (Pedersen 1987; Pedersen et al. 1989). It is not necessarily the landslides themselves that can be disastrous, but the tsunamis related to mass movements out into the deeper parts of the sea. These dangerous landslide and tsunami events appear with a frequency of about once every 50 years.

The latest event occurred in November 2000 (Pedersen et al. 2002): the coast at the settlement Saqqaq was flooded by a series of giant waves triggered by a 90 million m$^3$ landslide from a cliff (1400 m above sea level). About 30 millions m$^3$ flowed seawards triggering the tsunami. The velocity of the tsunami was calculated to 240 km/h. Ten boats were destroyed, but luckily no lives were lost.

REFERENCES


SLOPE SAFETY AND LANDSLIDE RISK MANAGEMENT IN KOREA

S. G. Lee  
Department of Civil Engineering  
The University of Seoul, Seoul, Korea

S. R. Hencher  
Halcrow China Ltd.  
Department of Earth Sciences, University of Leeds, UK

Abstract: This paper describes the characteristics of landslides in South Korea and the ways in which the many cut slopes are managed by various Authorities. The problems of assessing slope stability through inspection and ground investigation are discussed and a plan for improved management is presented. The paper is illustrated with numerous case studies of slope failures.

INTRODUCTION
In this paper aspects of natural landslides and cut-slope failures in Korea are introduced and the current situation regarding slope management is discussed. Problems associated with investigation and design have been identified through a review of standards and reports on cut-slope construction in Korea. The need for improved standards for slope construction and maintenance in Korea and for an integrated slope management system are addressed through analysis of case examples. It is concluded that many failures can be attributed to poor understanding of geological factors and related conditions.

KOREAN CONDITIONS
South Korea is a peninsula located in north-eastern Asia and situated between China and Japan, with an area of 99,600 km$^2$. The capital is Seoul and the population is about 50,000,000. In general, the peninsular is mountainous (about 70% of the total area), but rarely exceeding 1,200m in altitude, with northeast-southwest trending physiographic features (Figure 1a). The mean annual temperature is 10$^\circ$C, ranging between -15$^\circ$C in winter and 30$^\circ$C in summer with four distinct seasons. The average annual rainfall is about 1200 mm, 60% of which falls between June and August (Figure 1b). Vegetation cover is about 70% of the total area of Korea.

The geology of the Korean peninsula consists of rocks ranging from the Precambrian to the Cenozoic era. Thirty percent is underlain by igneous rock, twenty five percent sedimentary and forty five percent metamorphic. There are no glacial deposits and the predominant soils are derived from in situ weathering (Figure 1c). Regardless of soil types in Korea, the depth of residual soils is generally limited to a few metres (Lee & deFreitas 1989). The rate of erosion, which occurs mainly during seasonal heavy rainfall from June to September, ranges from 15 to 20mm per year on hillsides with inclination of 20$^\circ$ to 30$^\circ$ (Figure 2). There is no recorded case of slope failure triggered by earthquakes in Korea (Lee 1993).
CHARACTERISTICS OF SLOPE FAILURE
In Korea, most landslides are triggered by intense rainfall. Risk to life and property is increasing as more development takes place on sites close to steep natural slopes in mountainous areas. Figure 3 illustrates typical modes of failure in Korea.

Life Casualty Status by Years
On average 60 lives, damage to property valued at 500~1,000 million US dollars and considerable traffic disruption can be attributed to landslides annually in Korea. This is a high proportion of damage from all natural disasters (10 to 20%). Despite such damage and rate of casualties (Figure 4) no single agency has been set up in Korea to manage slopes on a par with the Geotechnical Engineering Office (GEO) in Hong Kong (Lee 2002).
Rainfall Characteristics of Korea

Slope failures in Korea are mostly associated with typhoons and seasonal rain fronts from June to September each year (Figure 5). The rainfall criteria used by the National Emergency Management Agency (NEMA) for warning against landslide risk are presented in Table 1. Paths of typhoons impacting Korea between 1950 and 2006 are illustrated in Figure 5. Analysis of rainfall triggering 100 failures from 1990 to 2005 shows that more than 80% of the failures were associated with maximum daily rainfall of up to 250mm with antecedent rainfall up to 400mm (Figure 6c) (Lee et al. 2007d).
Table 1: Criteria for landslide warning signals in Korea (NEMA 2005)

<table>
<thead>
<tr>
<th>Classification</th>
<th>Landslide alert (mm)</th>
<th>Landslide warning (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Antecedent precipitation (2 days)</td>
<td>100~200</td>
<td>&gt; 200</td>
</tr>
<tr>
<td>Daily precipitation</td>
<td>80~150</td>
<td>&gt; 150</td>
</tr>
<tr>
<td>Hourly precipitation</td>
<td>20~30</td>
<td>&gt; 30</td>
</tr>
</tbody>
</table>

Figure 5: Typhoons impacting Korea between 1950 and 2006

Figure 6: Korea’s typical precipitation pattern. (a) Monthly mean precipitation from Jan 1961 to Sept 2006 (KMA 2006), (b) Maximum precipitation and duration from 1976 to 2006 (KMA 2006), (c) Correlation between daily and accumulative precipitation (Lee et al. 2007d).

Landslides in Natural Terrain
Examples of typical landslides are shown in Figure 7. Landslides generally occur in hill slopes inclined at 20° to 40°. Most hill slope failures are relatively small scale with run out length of up to 20 to 30m, and width less than 10m. Failure depth is typically less than 1m. The angle of landslides along valley sections is 30° on average, with various lengths from 100 to 500m. Considerable damage is associated with erosion along such watercourses (Figures 8 to 10) (Lee 1987; Lee 1988a; Lee 1994; Lee et al. 2007d, 2008c; Shin 2009).
Figure 7: Typical landslides in Korea (July 2006). (a) Hill slope failure involving sliding of shallow soil horizon on rock, (b) Intense landsliding in lower valley sides, (c) Debris flow along river.

Figure 8: Landslides caused by intense storm in 2006 (hourly maximum 70mm, daily maximum 202mm, accumulative 630mm over 6 days). (a) Satellite image taken after failure, (b) Aerial photograph of landslide locations, (c) General and channelized landslides.

Figure 9: Landslide damage in Korea. (a) Schematic diagram of landslide damage (Lee et al. 2007d), (b) Area damaged by landslides from 1976 to 2005 (KFS 2006).
Cut-Slope and Retaining Wall Failures

Cut-slopes are not generally regarded as important structures in Korea. Whilst the safety of tunnels and bridges are managed systematically, this is not the case for cut-slopes. Slope stability is often only assessed using simple data sheets or check lists following the standards of independent management agencies such as those for some highways, national routes and railways. Examples of typical cut-slopes and their failure are illustrated in Figure 11.

DOMESTIC SLOPE MANAGEMENT

Management Status by Agencies

Landslides Management Status

Landslides are being studied by 3 main agencies in Korea namely KFRI (Korea Forest Research Institute) (Figure 12), KIGAM (Korea Institute of Geoscience and Mineral Resources) and NIDP (National Institute for Disaster Prevention). The current situation is summarized in Table 2.
Table 2: Landslides management institutions (KFRI 2000; KIGAM 2006; NIDP 2005)

<table>
<thead>
<tr>
<th>Name of institution</th>
<th>Name of systems</th>
<th>Study period</th>
<th>Study Output</th>
<th>Contents</th>
</tr>
</thead>
</table>
| KFRI                | Management system using landslide hazard map | 1980 to present | -Database of 2,295 landslides  
-Landslide hazard map of entire nation (1:25,000, 750 drawings) | Landslide hazard map on the web  
http://www.kfri.go.kr |
| KIGAM               | QRA (Quantitative Risk Assessment) | 1996 to present | - Database of 4,500 landslides  
-Partial landslide prediction map for 22 mountain areas | Web service is scheduled  
http://www.kigam.re.kr |
| NIDP                | Policy study to reduce hazards from natural landslides & cut-slope failures | 1997 to present | - | Seven reports on natural terrain landslides and cut-slope failures  
http://www.nidp.go.kr |

Figure 12: Landslide hazard map of KFRI (http://www.kfri.go.kr)

**Cut-Slopes Management Status**

Following increasing criticism that cut-slope management in Korea is generally poor (MBC, 1997), the Korean government has begun to take more interest in the stability of cut-slopes. Since 1999, cut-slopes along the national route are managed by KICT (Korea Institute of Construction Technology) of MOCT (Ministry of Construction & Transportation), highway cut-slopes are managed by ETTI (Expressway & Transportation Technology Institute) of KEC (Korea Expressway Corporation) and railroad cut-slopes are managed by KRRI (Korea Railroad Research Institute) of KRC (Korea Railroad Corporation). Furthermore a “Special Law Related to Safety Management of Facilities” was introduced in 2003 and now large retaining walls are managed by KISTEC (Korea Infrastructure Safety and Technology Corporation). Since 2007, a plan to manage cut-slopes of all scales including any local
government and privately owned ones not covered by “The Law Related to Steep Slope Disaster Prevention” (NEMA) has been studied at the LRC (Landslide Research Centre). The management status of each agency is summarized in Tables 3 and 4.

Table 3: Cut-slope management institutions (KICT 2005; ETTI 2004; KRRI 2004; KISTEC 2003; Lee 2007)

<table>
<thead>
<tr>
<th>Name of institution</th>
<th>Management method</th>
<th>Study period</th>
<th>Study Output</th>
<th>Contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>KICT</td>
<td>CSMS (Cut-slope Management System)</td>
<td>1999 to present</td>
<td>-12,650 database of slopes &gt;6m along 14,000km of national route -3,144 precise database of risky areas</td>
<td>-Establishment of database for the priority of national route reinforcement -Installation of monitoring instrumentation at 66 locations</td>
</tr>
<tr>
<td>ETTI</td>
<td>HCMS (Highway Construction &amp; Maintenance System)</td>
<td>2000 to present</td>
<td>5,335 highway slopes of &gt;5m in database</td>
<td>-Integrated management of data related to highways -Installation of monitoring instrumentation at 13 locations</td>
</tr>
<tr>
<td>KRRI</td>
<td>IR-DiPS (Intelligent Railroad Disaster Prevention System)</td>
<td>2000 to present</td>
<td>38 database of slopes alongside 7 national railways</td>
<td>-Database of railroad slopes -Installation of monitoring instrumentation at 3 locations -Expecting to establish an automated system to predict and monitor failures</td>
</tr>
<tr>
<td>KISTEC</td>
<td>Special Law Related to Safety Management of Facility</td>
<td>2003 to present</td>
<td>178 retaining walls with height &gt;5m and width &gt;100m 169 retaining walls with height &gt;50m and width &gt;200m</td>
<td>-2 checks each year -1 detailed check every 2 years</td>
</tr>
<tr>
<td>LRC</td>
<td>The Law Related to Steep Slope Disaster Prevention, Slope hazard map</td>
<td>2006 to present</td>
<td>database of 12,899 cut-slopes by local governments 10,400 borehole database and ground information system for Seoul</td>
<td>-Research into slope design according to rock types</td>
</tr>
</tbody>
</table>

http://www.kict.re.kr
http://research.freeway.co.kr
http://www.krri.re.kr
http://www.kistec.or.kr
http://www.cut-slope.re.kr
Table 4: Korea’s current road conditions and the number of slopes under control

<table>
<thead>
<tr>
<th>Division</th>
<th>Total length (km)</th>
<th>No. of managed slopes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total No. of roads</td>
<td>102,293</td>
<td>18,023</td>
</tr>
<tr>
<td>National highways</td>
<td>3,103</td>
<td>5,335</td>
</tr>
<tr>
<td>National roads</td>
<td>14,225</td>
<td>12,650</td>
</tr>
<tr>
<td>National railways</td>
<td>3,380</td>
<td>38</td>
</tr>
<tr>
<td>Local government routes</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roads of metropolis</td>
<td>17,738</td>
<td></td>
</tr>
<tr>
<td>Country Roads</td>
<td>17,677 (3,673)</td>
<td></td>
</tr>
<tr>
<td>City roads</td>
<td>25,360</td>
<td></td>
</tr>
<tr>
<td>Province roads</td>
<td>23,958</td>
<td></td>
</tr>
</tbody>
</table>

**Failure of Slopes Under Management**

Despite the fact that KICT has been managing slopes along national roads systematically since 1999, the reality is that the number of failures per year has not been significantly reduced (Figure 13). Furthermore some slopes that had been identified as low risk have collapsed resulting in a number of human casualties. Figure 14 shows the result of the failure of a 10 metre high cut-slope on a national road. Ten cars were buried and 4 people died as a consequence.

![Figure 13: Slope failures adjacent to national roads (MOCT 2006)](image-url)
Slope Stability according to Agencies and Analysis of Table used for Assessment

A slope safety assessment table used in Korea has similar categories for assessment and scoring factors as that used in Hong Kong. Comparative analysis of failed slopes at 50 locations, using the assessment table of GEO (Watkins & Koirala 1986) and those of Korean agencies such as KICT (2005) and ETTI (2004), is shown in Figure 15. The GEO table has rather better predictive capabilities for estimating risk in terms of entire slope risk rating given the high emphasis on slope height and angle. By comparison, the assessment method used in Korea has a wider score distribution and is rather more subjective (Lee et al. 2007d).

Slopes under Jurisdiction of Local Governments

It is estimated that there are about 1 million cut-slopes in Korea which is of the order of 20 times the number in Hong Kong (Lee et al. 2008b). Research has not been conducted nor slope management systems implemented as yet for most slopes under the jurisdiction of local governments. There is therefore an inaccurate knowledge of the status of slopes nation-wide.

A survey has been carried out for the entire country by NEMA and by LRC (the Landslide...
Research Centre) to identify current conditions of slopes under local government ownership and privately owned. 12,900 slopes have been identified and risk analysis carried out using the slope information obtained to date (Figure 16) (Lee et al. 2008b).

Figure 16: Cut-slope distribution and reported damage in Korea (Lee et al. 2008b). (a) Estimated cut-slope concentration in Korea, (b) Active development in mountainous areas, (c) Assessed risk distribution.

Current construction laws stipulate that houses should be constructed at a distance from a slope as great as the height of any adjacent cut-slope or retaining wall. However in reality many houses are being built closer to cut-slopes in mountainous areas because authorizing bodies allow exceptions to the rule (Figure 17).

Figure 17: Regulations on building construction concerning cut-slopes. (a) Building construction regulation details (MOCT 2007), (b) Houses being constructed close to cut-slopes in Seoul (2002).
Rock Treasures
Korea has many “rock treasures” designated for preservation, but there are many cases of serious damage and rock collapse at such locations due to a lack of appreciation of engineering geological conditions and hazards (Figures 18 & 19) (Lee 2004).

Figure 18: National rock treasure (rock carving) failure caused by tree roots (Treasure No. 221, 2003; A.D.670). (a) Photograph taken before failure, (b) photograph taken after failure, (c) Opening of vertical joints caused by tree root jacking, (d) Schematic diagram.

Figure 19: Bankudae petroglyphs engraved between B.C. 3500 and 3000 (National Treasure No. 285). (a) General view of petroglyphs and location of Schmidt hammer rebound tests, (b) Shallow hornfels layer on petroglyphs surface, (c) Micro-cracks opened up by Schmidt hammer tests, (d) Inundation by water, (e) Exposure during the dry season.

REALITIES OF CONSTRUCTION AND MANAGEMENT FOR CUT-SLOPES
A flow chart illustrating the process of slope investigation, design, construction and maintenance in Korea is presented in Figure 20.
Figure 20: Flowchart illustrating process of slope investigation, design, construction and maintenance in Korea.

Realities of Applying Slope Design Standards
Although Korea has standards for the survey and design of slopes developed by individual agencies, the geological input to design tends to be poor. Following a review of 100 design reports, less than 10% were identified as having a sufficient geological input for design (Lee 2007d). It is the opinion of the authors that lack of geological input and lack of geological understanding in slope design may account for between 30% and 50% of failures that take place during slope construction (Lee 2007). In one case a slope failed six times at the same place due to improper design and safety inspections after each collapse (Lee & Hencher 2007).

Problems regarding Geological Survey and Ground Models

Borehole Investigation
The standard in Korea is to drill to a level 2m deeper than that where moderately weathered rock is first encountered. In many cases borehole data very remote from the location of the slopes is employed as if it were representative of the slope and in many cases geology is determined from only one borehole (Figure 21).
Figure 21: Problems with borehole investigations. (a) Locations of boreholes too far from cut slope and therefore irrelevant, (b) Reliability of model depends on the depth of boreholes: Case ① - investigation taken to design height +2m (OK); Case ② - borehole only taken to “soft rock” boundary and therefore important structure not identified.

Ground Modeling of Slopes
Government organization such as KEC (2001) and MOCT (2007) provide Korean construction standards as a guideline for the design of cut-slopes as follows: moderately weathered to fresh rock 1:0.5 (63°), highly weathered rock 1:1 (45°), and soil 1:1.2 (40°) to 1:1.5 (34°) depending on rock material strength described in Table 5. However this approach has difficulties. For example the design for a moderately weathered rock mass is based solely on the recovery of moderately weathered rock fragments during borehole investigation and this leads to inaccurate ground modeling (see Figure 22 for example). A new slope standard that incorporates TCR(%) and RQD(%) has been proposed by Lee (1991) and Lee and Lee (1995) and has been widely accepted by many government organization such as KEC (2001) and MOCT (2007) but still this is rather questionable for design due to a lack of proper interpretation of ground conditions from this basis alone (Table 6) (Lee & Hencher 2007).

Figure 22: Examples of different interpretations of the same drill logs. (a) Based on rock material classification, (b) Based on rock mass classification.
### Table 5: Rock classification for excavation purposes widely used in Korea (CAOK 2001)

<table>
<thead>
<tr>
<th>Classification factor</th>
<th>Group of rock</th>
<th>Seismic wave velocity of the ground $V$ (km/sec)</th>
<th>Seismic wave velocity of rock sample $V_c$ (km/sec)</th>
<th>Uniaxial compressive strength of intact rock (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weathered rock</td>
<td>A</td>
<td>0.7 to 1.2</td>
<td>2.0 to 2.7</td>
<td>30 to 70</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>1.0 to 1.8</td>
<td>2.5 to 3.0</td>
<td>10 to 20</td>
</tr>
</tbody>
</table>
| - Heavily weathered rock with 1 to 10 cm joint spacing.  
- Pickaxe or some blasting is required for excavation of the rock |
| Soft rock             | A             | 1.2 to 1.9                                    | 2.7 to 3.7                                       | 70 to 100                         |
|                       | B             | 1.8 to 2.8                                    | 3.0 to 4.3                                       | 20 to 50                          |
| - Rock with 10 to 30 cm joint spacing.  
- Blasting is needed for excavation or cutting of the rock. |
| Medium rock           | A             | 1.9 to 2.9                                    | 3.7 to 4.7                                       | 100 to 130                        |
|                       | B             | 2.8 to 4.1                                    | 4.3 to 5.7                                       | 50 to 80                          |
| - Slight sign of weathering; 30 to 50 cm joint spacing.  
- Blasting is needed for excavation or cutting of the rock. |
| Hard rock             | A             | 2.9 to 4.2                                    | 4.7 to 5.8                                       | 130 to 160                        |
|                       | B             | > 4.1                                         | > 5.7                                            | > 80                              |
| - Rock with 50 cm to 1 m joint spacing.  
- Blasting is needed for excavation or cutting of the rock. |
| Very hard rock        | A             | > 4.2                                         | > 5.8                                            | > 160                             |
| - Very strong rock with wide joint spacing over 1 m. |

Group A: gneiss, sandy schist, green schist, hornfels, limestone, sandstone, diabase tuff, conglomerate, granite, diorite, peridotite, serpentine, rhyolite, andesite, basalt

Group B: dark schist, green schist, diabase tuff, shale, mudstone, tuff, agglomerate

### Table 6: Standard gradients for cut slopes occasionally used in Korea (Lee 1991; Lee and Lee 1995; followed by KEC 2001 and MOCT 2007)

<table>
<thead>
<tr>
<th>Types of rock masses (strength &amp; fracturing)</th>
<th>Degree of fracturing in rock masses</th>
<th>Excavatability</th>
<th>Gradient</th>
<th>Shear strength parameters of rock masses</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>NX borehole size (using double core barrel)</td>
<td>TCR</td>
<td>RQD</td>
<td>phi (degree)</td>
</tr>
<tr>
<td>Highly fractured rock mass</td>
<td>&lt;20%</td>
<td>&lt;10%</td>
<td>Rippable rock mass</td>
<td>1:1.0 (45°)</td>
</tr>
<tr>
<td>Less fractured MW to F rock mass</td>
<td>20 to 30%</td>
<td>10 to 25%</td>
<td>Excavable rock mass (HW to MW rock mass)</td>
<td>1:0.8 (51°)</td>
</tr>
<tr>
<td></td>
<td>40 to 50%</td>
<td>25 to 35%</td>
<td>MW to SW rock mass</td>
<td>1:0.7 (55°)</td>
</tr>
<tr>
<td></td>
<td>&lt;70%</td>
<td>40 to 50%</td>
<td>Fresh rock</td>
<td>1:0.5 (63°)</td>
</tr>
</tbody>
</table>
**Determination of Method for Excavation**

There is no clear standard for rock classification for ripping and blasting rock and for the assessment of excavation difficulty in Korea and disputes with respect to excavation costs are commonplace (Figure 23). Incorrect decisions are sometimes taken due to the indiscriminate application of rock classification standards (Figure 24) (Franklin 1971; Lee et al. 2007b).

---

**Figure 23:** Rock mass classifications for excavation purpose based (a) on uniaxial compressive strength only and (b) on both uniaxial compressive strength and fracture intensity.

---

**Figure 24:** An example of the difficulties in the use of rock mass classification for excavation purpose (Lee et al. 2007b). (a) quartzite with micro-cracks and incipient joints as well as more persistent open joints b) The decision re Zone ② or Zone ① (Rippable or blasting required) depends on how the fractures are considered which is a matter of judgment.
The seismic method for estimating excavatability is relatively accurate for gradually varying weathering profiles in Korea but its accuracy is reduced in cases where corestones are developed (Lee & deFreitas 1990; Lee et al. 2007c). In such cases excavatability must be determined by considering all aspects of the rock mass that constrain workability (Figure 25).

![Figure 25: Evaluation results of excavatability of weathered profile with corestones (Lee et al. 2007c).](image)

Problems of Slope Stability Analysis Method
For slope stability analysis, shear strength values are commonly adopted following literature review rather than laboratory tests. Similarly single sets of parameters are used assuming isotropic and homogeneous conditions despite complex geology with, say, the potential for failure along relic joints, sliding along the boundary between soil and rock or failure through variable corestone weathering profiles. Such possible failure modes are rarely considered sufficiently at the design and safety inspection stages.

Stereographic analysis is routinely used for the design of rock slopes. However joint networks are often analysed over-simplistically without considering the actual geological conditions in the field, detailed joint properties and interrelationships between sets of fractures. This can result in failure as reported by Hencher (1987) and Hencher & Knipe (2008) and illustrated in Figure 26. Cases are known where failure has occurred because adverse joints, not considered for the original design, have been discovered during construction and the response has been too slow for the adoption of appropriate reinforcement measures (Lee et al. 2003).
Figure 26: Misuse cases of stereographic projection method. (a) Failure to consider effects of vertical release joints to the rear of kinematically adverse joints (Lee and Geum 2002), (b) no consideration of the location of discontinuities during stereographic analysis.

Standard factors of safety used for slopes in Korea are $F_s > 1.5$ for the dry season and $F_s > 1.2$ for the wet season. Although there are no reports of slope failure due to earthquakes in Korea, there are cases of slopes being designed with $F_s > 1.1$ to account for potential earthquake shaking (KEC 2001).

Problems of Construction Method and Maintenance
The deployment of uniform fences along road-side slopes, installation of wire-mesh, rock bolts and anchors, excessive reinforcement of stepped buttresses, poor drainage design of nailed slopes and installation methods of concrete drainage measures can all be potential sources of problems in Korea as illustrated in Figure 27.

Systemic Problems
Systemic problems in Korea include a lack of detailed guidelines and standards for geological survey and for the design and maintenance of slopes. It is recognised that there must be a balance in the preparation of guidelines in that that over-prescription may stifle professionalism but one outcome of a lack of standards is that it is sometimes difficult to identify responsibility for failures. A more specific problem is that data on slope construction is only legally required to be retained for up to 5 years. It is the opinion of the authors that there is a need to create a database for ongoing management of slopes in Korea linked to guidelines on deterioration and maintenance.
Finally it is considered that the Korean education system fails to provide adequate training for many geotechnical engineers in the important areas of engineering geology and soil and rock mechanics.
Figure 27: Reinforcement and maintenance issues. (a) Uniform wire-mesh and fence, (b) Excessive anchoring, rock bolts and stepped buttress, (c) Insufficient drainage in conjunction with soil nailing, (d) Frozen cut-slope, (e) Concrete drainage in valley unsuitable for the geological conditions, (f) Lack of identification of imminent slope failure despite real-time monitoring.

CASES OF SLOPE FAILURES
Analysis of major cases of cut-slope and retaining wall failures post construction indicates that the main causes are the lack of adequate geological survey and inadequate drainage (Lee et al. 2007b).
Slope Failures Characteristic of Particular Rock Types

A number of typical examples are discussed below grouped according to rock type (Figure 28):

**Igneous Rock**

Igneous rocks are evenly distributed throughout 30% of Korea; most are plutonic.

**Plutonic Rocks**

Most of the plutonic rocks are granites which were intruded at depths greater than 2km from the ground surface during the Jurassic and Cretaceous periods. Weathering is generally not deep compared to more tropical areas and profiles are gradational, with the development of corestone weathering locally. Certain profiles may deteriorate rapidly in engineering time on exposure (Lee 1988b; Lee et al. 2006). Vertical and sheeting joints are commonly developed and the sheeting joints in particular have an important effect on slope stability (Figures 29 & 30).

---

**Figure 28: Locations of 18 case studies**

**Figure 29: Engineering geological characteristics of granites in Korea. (a) Sheeting joints, (b) Vertical joints, (c) Gradational weathering profile, (d) Corestone weathering Profile.**
Case 1: Persistent sheeting joints

The stability of a cut-slope in moderately to slightly weathered granite in Seoul constructed at 1:0.3 (73°) has been investigated because of repeated failures. The failures were shown to be associated with the presence of sheeting joints and as a result the lower slope had to be redesigned at 55° with the upper slope at 35° (Figure 31) (Lee 1999).

Case 2: Impersistent sheeting joints

An extensive 80m high rock slope was under construction in Seoul alongside an express highway. The original design gradient of 1:0.5 was reduced to 1:1 during construction because of concerns over risk to the highway. Additional geological mapping during construction identified numerous sheeting joints that might allow planar failures according to their orientation. The potential for large scale failure along the sheeting joints was reviewed.
in detail but considered unlikely because their persistence was only of the order of 0.5 to 1m. Faults with persistence of 15 to 20m were discounted because they were judged unable to contribute to failure (Figure 32) (Lee 1998a; Lee et al. 1991).

Case 3: Continuous Failure of Granite Slope with Corestones
The cut slope is located alongside a national highway and consists of granite with a corestone weathering profile and many faults. This slope was originally considered to comprise uniform moderately weathered rock on the basis of seismic survey without boreholes and designed to a gradient of 1:0.5 and height of 50m (3 million US dollars in construction costs). Joint directions were measured but there was no detailed face mapping. The 1st failure occurred during construction and the slope was re-designed with a gradient of 1:1 and a slope height of 130m (6 million US dollars in construction costs) following stereographic projection analysis (KEC 2000). A 2nd failure occurred during re-cutting and the slope design was again reduced to a gradient of 1:1.2 and proposed cutting height of 200m following surface mapping and stability analysis. In the event a rock shelter tunnel (5 million US dollars in construction costs) was adopted instead as the final design because there was a concern of excessive environmental damage. In summary the original construction cost of 3 million US dollars increased to 14 million US dollars together with considerable environmental damage caused by unnecessary excavations. This situation could be attributed to a lack of detailed geological investigation and analysis at the design stage (Figure 33) (Jeong et al. 2001; Lee et al. 1991).
Figure 33: Collapse of granite cut-slopes in Yeongdong highway between Wonju and Gangneung of Kwangwon province. (a) Safety checks after the first collapse in November 1999, (b) The second failure in May 2001, (c) Construction of reinforced concrete rock-shed at the bottom of cut-slopes.

**Volcanic Rocks**

Volcanic rocks are mainly found in the south of the Korean peninsula (Figure 31). Such rocks often have few joints although columnar joints are sometimes present and are associated with rock falls. In terms of weathering, there is sometimes an uneven, inter-bedded weathering characteristic. Thin residual soil of 1m or less occurs at the ground surface and further uneven weathering occurs beneath lava flows. Lava tunnels are sometimes seen (Figure 34).
Case 4: Columnar Jointed Andesite

Mt. Sanbang is a famous tourist location at the southwest of Jeju Island to the south of the Korean peninsula. It is a dome shaped area with slopes of 70° to 80° and height of 50m formed by columnar jointed andesite. Slope safety assessments have been carried out because of numerous rock falls from a 400m long cliff posing a risk to a road at the foot of the mountain.

The rock fall hazard has been assessed by surface geological mapping, 3D surface scanning and numerical analysis using the distinct element method. Following aerial photograph analysis, topographical analysis and rock fall energy analysis, catch nets have been established in sections at the bottom of the cliff (Figure 35) (Lee et al. 2007a).

Figure 35: Treatment of andesite natural slope with many columnar joints adjacent to national road at Mt. Sanbang in Jeju island. (a) Planned catch nets against overall view of Mt. Sanbang and lower part of cliffs, (b) Discontinuity surfaces identified using 3D scanner, (c) Analytical results of slope safety, (d) Identification of hazardous zones through aerial photograph analysis, (e) Expected pathway prediction of falling rock debris through 3D geomorphological analysis, (f) Energy analysis for net design.
**Metamorphic Rocks**

Metamorphic rocks are distributed throughout 45% of Korea. Metamorphic rocks tend to have relatively impersistent joints but are associated with many slope failures. Weathering tends to be relatively deep and irregular and corestone weathering is locally developed (Figure 36).

![Figure 36: Engineering geological characteristics of metamorphic rocks in Korea. (a) Faults, (b) Joints, (c) Irregular weathering, (d) Corestone weathering.](image)

**Case 5: Consecutive Failures along a Single Discontinuity**

The slope illustrated in gneiss has suffered three consecutive wedge failures involving one extensive surface with fault gouge. Because of pervasive fracturing it is found that failure is progressive. To prevent such on-going deterioration, prompt investigation and remediation are necessary (Figure 37) (Lee 1998a).

![Figure 37: Consecutive failures along faults with fault clay in gneiss in road cut during construction. (a) Three consecutive failures in 1993, (b) Opening of discontinuities after failure.](image)
**Case 6: Failure along Persistent Joints in Quartzite**
Quartzite commonly has systematically developed joints with persistence over 10m and there many failures occur during road construction (Case 6-1; Figure 38(a)) or during quarrying for aggregate production (Case 6-2; Figure 38(b)) (Lee 1998b).

![Figure 38: Failure along persistent quartzite joints. (a) Typical joints in quartzite, (b) Case 6-1: slope failure in quarry near Seoul, (c) Case 6-2: slope failure during excavation of lower parts of road cuts in Seoul.](image)

**Case 7: Revised Design to Account for Degree of Jointing in Quartzite**
The cut-slope was originally designed with anchors. However during excavation the cut-slope illustrated was re-designed to be cut steeply and less-reinforced with rock bolts based on the high strength of quartzite material. Further study reassessed that because the rock is highly fractured the slope would be unstable and was redesigned with many rock anchors (Figure 39).

**Case 8: Failure Slope in Gneiss with Corestone Weathering**
The slope was designed with a gradient of 1:0.5 (63°) assuming uniform moderately weathered rock judged from rock exposed at the ground without any borehole investigation. A circular-type failure occurred during excavation work and the gradient was modified to 1:1 (45°) after the failure. Then a 2nd circular-type failure occurred again during excavation, so a rock shed tunnel was built after a safety assessment (Figure 40) (Lee & Geum 2000).

**Sedimentary Rocks**
Sedimentary rocks make up 25% of South Korea; bedding tends to be gentle with gradients of about 10-20°. Even within a single cut-slope, various layers of rock may be present exhibiting different degrees of weathering and different strength characteristics. Limestone typically shows less discontinuities than other sedimentary rocks and thinly developed residual soil of less than 1m (Figure 41).
Figure 39: Consideration of both orientation and spacing of joints in quartzite road cut-slope of Seoul. (a) Original and revised designs based on the strength of rock material, (b) Vertical joints of long-length in quartzite observed during construction, (c) Numerical slope stability analysis in consideration of discontinuities, (d) Final reinforcement with anchor.

Figure 40: Failure of road cut-slopes in gneiss with corestone weathering. (a) Geological characteristics of failure areas, (b) Construction of reinforced concrete rock-shed at the bottom of road cut-slopes after consideration of various remedial works.
Figure 41: Engineering geological characteristics of sedimentary rocks in Korea.  
(a) Bedding planes, (b) Interbedded layers of strong sandstone and weak mudstone, (c) Less joints and thin residual soil on limestone.

**Case 9: Repeated Failures along Bedding**
In this case a decision had to be taken as to whether to adopt a tunnel or change the route to avoid slope problems after a series of failures in 1993, 1994 and 1995 even at a gentle slope of 25°. The original slope design was for a gradient of 1:0.5 (63°) without considering the possibility of failure related to bedding planes and uneven weathering despite exposures indicating the nature of the local geology (Figure 42) (KEC 1995).

Figure 42: Consideration of route changes due to repeated failures along bedding planes (a) Bedding planes and uneven weathering, (b) Route changes or tunnel construction.

**Case 10: Change of Design to Account for Weathering of Sandstone and Mudstone**
A partial failure occurred in the slope illustrated during construction. The original slope was designed with a slope gradient of 1:0.5 (63°) based on Korean construction standard because moderately weathered rocks were encountered near the ground surface during site investigation. Following a safety assessment, the slope gradient was reduced to 1:1 (45°) because of the presence of highly weathered mudstone and highly jointed sandstone near the slope toe (Figure 43) (Lee 1998a).
Figure 43: Slope stability inspection in sedimentary rocks showing differential weathering of sandstone and mudstone during the road cut construction. (a) Inclination of bedding planes, (b) Safety examination considering differential weathering.

**Case 11: Failure of Slope in Alternating Limestone and Mudstone at Tunnel Portal**
The upper slightly weathered limestone suffered a large scale failure while excavating highly weathered mudstone for a tunnel portal (Figure 44) (Lee and Son 2002). Failure occurred due to the presence of extensive vertical joints combined with gentle bedding of $10^\circ$.

Figure 44: Failure of slopes in alternating limestone and mudstone during construction at tunnel portal. (a) Overall view, (b) Development of discontinuities.

**Case 12: Reinforcement of Lower Section of Slope Following Deterioration of Mudstone Underlying Sandstone**
Mudstone underlying sandstone was eroded following long-term exposure and as a result an overhang was created and the bottom of slope needed to be supported with a concrete buttress (Figure 45) (Lee 1998a).
Case 13: Joint Development in Shale

The RQD of moderately to slightly weathered shale can be largely a matter of judgment depending on whether sections with weakly bonded joint planes between open, mechanical discontinuities are deemed to constitute “sound” rock or not. The implications of such decisions are illustrated in the case shown in Figure 46 (Lee & Son 2003).
**Case 14: Weathering Characteristics of Mudstone Slope**
The mudstone shown in Figure 47 is a Tertiary deposit. The upper section of mudstone was
gave Schmidt Hammer Rebound values of 10~18 during excavation; the lower slope gave
values in the range 26~36. Weathering has rapidly progressed post excavation to a depth of
300 to 500 mm from the slope surface. Numerical analysis of the slope based on conditions in
the exposed rock suggests that a cut with gradient of 1:0.3 (73°) might be unstable but in fact
the overall slope is actually quite stable because of the controlling influence of the less
weathered rock at depth although surface raveling is still a problem (Figure 47) (Lee et al.
2006).

Figure 47: Development of weathering in mudstone cut-slopes. (a) Front view;
highly fractured at the upper slope excavated in 1996 and relatively
massive at the lower slope excavated in 2005, (b) Side view showing the
development of weathering penetrating 300~500 mm from the cutting
face after 10 years, (c) Slope stability analysis indicates the existing
stable slope of 1:0.3 (73°) to be unstable even under dry conditions.

**Case 15: Failure along a Fault in Conglomerate**
While excavating a slope in conglomerate for a coal mine railway, a failure occurred along a
fault with clay infill (Figure 48) (Lee 1988b).
Complicated Geological Conditions
Where the geological situation is complicated with mixed rocks and complex structures, each case must be considered individually.

Case 16: Stability Analysis of Mixed Rock Masses
The area comprises sedimentary rocks with irregular intrusions of quartz porphyry. The stabilizing effect of the intrusions was considered in detail within individual slices through the slope (Bishop 1955) to assess the need for preventative measures (Figure 49) (Lee & Hencher 2008).

Case 17: Slope Stability Analysis for a Fault Zone
Stability analysis was carried out for a slope constructed at 1:0.5 after recognising a complicated fault zone during excavation. Rock strength and fault geometry were established as a priority. The risk of failure due to interaction between various faults was confirmed and, because of the high possibility of wedge failure, the slope was regraded and reinforced by rock bolts in some zones (Figure 50) (MOCT 1997).
Case 18: Failure along the Boundary between Soil and Rock

The bottom of the slope has been supported with gabions because a failure occurred at the boundary between soil and rock during excavation (Figure 51) (Lee et al. 2008a).

Figure 49: Slope stability analysis of a complex rock mass of sandstone intruded by irregular quartz porphyry dykes. (a) Failure along a bedding plane during excavation of the right-upper part of the road cutting (b) Schematic diagram of (a), (c) Cross-section showing the method of numerical stability analysis (Sta. 1+200), (d) Results of the numerical stability analysis.
Figure 50: Stability analysis in zone with numerous faults during road cut construction. (a) Overall views of slopes, (b) Results of face mapping, (c) Statistical stability analysis, (d) Recommended remedial works.
Failure of Areas Adjacent to Cut-Slopes
There are many cases of damage caused by slope failure, debris flow and rockfall from areas adjacent to cut-slopes.

Failure of Upper Cut-Slope Area
Occasionally the toe area of a cut slope is impacted by failure of the natural slope above the cutting (Figures 52 & 53).

Figure 52: Traffic disruption caused by upper natural slope failure in Donghae highway of Gangwon province in August 2002. (a) Plan of failure scene, (b) Emergency restoration, (c) Upper natural slope failure.

Figure 53: Failure of upper part of cut-slopes above national roads in July, 2006. (a) Hangye-ryeong in Gangwon province, (b) In-je in Gangwon province.
Landslides in Upper Areas of Slope Affecting Lower Slopes

Traffic disruption occurred from debris flows at valley outlets impacting a highway near cut areas in 2002 and 2006 and then again in 2007. Part of the problem is that different management agencies are responsible for the cuts and upslope areas and this has led to inadequate planning and remediation measures (Figures 54 to 56).

Figure 54: Traffic interruption caused by debris flow around Donghae highway of Gangwon province in Sept 2002. (a) Distant view of landslide, (b) Debris flow from valley, (c) Bus buried under debris, (d) Traffic interruption for 3 days.

Figure 55: Debris flow in valley area disrupted transportation in Yeongdong highway of Gangwon province for 3 days (July 2006) (a) Satellite photograph (Google Earth 2007), (b) Landslide along valley, (c) Debris flow and cars buried by trees, (d) Damage to large bus, (e) Disruption of transportation, (f) The same site controlled by different authorities, (g) Improperly constructed horizontal debris barrier, (h) View across inadequate horizontal debris barrier.
Figure 56: Tunnel closed for two days because of debris flow blocking tunnel entrance from valley of upper slopes of Gwangchi tunnel in Gangwon province in July 2007. (a) Satellite photograph (Google Earth 2007), (b) View of collapsed area, (c) Emergency remedial works.

Damage to Lower Areas from the Failure of Forest Road Slopes
While constructing forest roads in a mountainous area, a debris flow occurred following the failure of cut-slopes resulting in burial of houses and destruction of a bridge at the lower section (Figure 57).

Figure 57: Debris flow from failure of the upper slopes of a forest road in Sancheong of Kyeongsang province in 2003. (a) Aerial photograph before failure, (b) Aerial photograph after failure, (c) Failure starting point at cut forest roads, (d) Damage by debris flow in lower part.
**Rockfall impacting Lower Areas**

Although there are many cases of damage caused to road and houses due to rockfall in mountainous areas remote from the facilities, it is difficult to establish a protection plan because the management agencies for the slopes and facilities are different (Figure 58).

![Damage and risk from rockfall in upper part of slopes](image)

Figure 58: Damage and risk from rockfall in upper part of slopes. (a) Part of national road damaged by rockfall from upper part of slopes within Sulak mountain national park in Gangwon province in July, 2006, (b) Dangerous rock in Bukhan mountain national park located above residential area in Seoul.

**FUTURE OBJECTIVES**

NEMA is a government agency and was established in 2004. In 2007 the “Law Related to the Prevention of Steep Slope Disasters” was introduced to help reduce human casualties and economic loss in relation to slope failure. Research related to this law is being performed by the LRC (Landslide Research Centre). The following aspects are being studied through a research project entitled “Prediction of Slope Failure and Development of Slope Management Technology.”
Slope Hazard Map and Korean Slope-hazard Management System (KSMS)  
A slope hazard map is being developed in Korea; this will help to identify areas of Seoul that are both mountainous with highly susceptible slopes and with crowded housing. Many houses have been (and may be) built in hazardous areas (Figure 59). In addition in the Seoul area, 10,300 boreholes have been incorporated into a database as part of a ground information management system. Other documents related to slopes have been included into a database to build a KSMS to be used for slope disaster prevention on a real-time basis (Figure 60).

Figure 59: Slope hazard pilot map targeting Seoul mountainous areas (Lee et al. 2008b). (a) Topography and geological map of Seoul, (b) Satellite image of Seoul, (c) Active development of housing in mountainous areas, (d) Slope disaster hazard map applied to mountainous areas, (e) The rise of housing development in steep mountainous areas with high susceptibility to disaster (distribution of areas for housing development between 1994 and 2002).
Korean Slope Classification System (KSCS) and Development of Equipment

Following a similar approach to as the RMR and Q-Systems for engineering work, five factors have been identified to develop a rock mass classification method for ease of planning slope construction. The new proposed method is called the KSCS (Korean Slope Classification System) and is a Korean investigation guideline and slope classification system appropriate for different rock types. Its purpose is to help civil engineers in performing investigation, design, construction and maintenance (Figure 61). In support, experimental and survey equipment are being developed. New multipurpose direct shear apparatus is being developed for testing boundaries between soil and rock (and intact soil and rock joints) whilst measuring both pore water pressure and pneumatic pressure. A 3D rock mass classification instrument that can measure discontinuity characteristics and degree of weathering at the same time remotely is being developed as well as automatic inclinometers.

Figure 61: Development of KSCS (Korean Slope Classification System) based on geological characteristics (Lee et al. 2008b).
CONCLUSIONS
Characteristics and causes of typical slope failures in Korea have been analysed and presented. It is noted that, despite the application of independent standards of various management agencies, a considerable number of failures have occurred associated with rapid industrialization. It is considered that this is partly because standards for slope design and management have not been well developed. The establishment and management of organizations to perform integrated management of slopes and standardized criteria (investigation, design, and construction technology) are necessary in order to minimize future slope damage. In 2006, NEMA of MOGAHA (the Ministry of Government Administration and Home Affairs) has commenced a study into slope failure prediction and the technology for reducing the risk from slope failures to below 70% of the 2006 level by 2016. This is being conducted, as a national study, to provide an integrated approach to the management of various slope types that are currently the responsibility of various agencies. Part of this initiative has been the establishment in July 2007 of “The Law Related to the Prevention of Steep Slope Disasters”. The establishment of an integrated management system to support this initiative requires continuing research.

REFERENCES


ACKNOWLEDGEMENTS

The Authors would like to thank Prof. Andrew Malone, Dr. Jimmy J. Jiao, Prof. David N. Petley, Dr. Robert Hack, Dr. Mike H. de Freitas, Dr. Derek Lichti, Prof. Soo-Young Lee, Prof. Ta Duc Thinh, Prof. Nie Lei for their assistance in field work and co-operation and Korean geotechnical engineers who reviewed this paper. This research was supported by a grant (NEMA-06-NH-05) from the Natural Hazard Mitigation Research Group, National Emergency Management Agency (NEMA), Korea.
LANDSLIDE RISK MANAGEMENT IN FRANCE: PRINCIPLES, ORGANISATION AND CHALLENGES

Eric Leroi
URBATER

Abstract: Since the beginning of mankind, there has been a need for sheltering, moving, building and development. Man has been learning to control the natural environment and then to apparently dominate it. In doing this, his relationship to the environment also changed. With the aid of technology, man did not have to try to fit into the natural environment any more, instead he worked it, modeled and manufactured it so that the environment will adapt to him, which is not achieved without some consequence to the equilibrium and the existing ecosystems.

Up to now, about 7,000 French “communes” are threatened by ground movement risks, of which a third are at a serious level in terms of population. Most of them are located in mountainous areas and exposed to various phenomena associated with slope and cliff instability (landslides, collapses of retaining walls, rock falls, etc.). Other communes located on flat plains or plateaus are affected by potential ground movements due to uncontrolled exploitations or dissolution of the ground.

Those ground movements are generally of short duration, focused phenomena of limited extent and limited effects. By their nature, together with their frequency and wide-ranging geographical distribution, they are nevertheless responsible for severe damages and significant and expensive detriments.

Landslide risk is managed in France in accordance with the general policy of natural risk prevention since the enactment of a law in 1982. However, this has not always been effective. Landslide risk management, its principles and its organization evolved with time along with the increasing social demand for enhanced protection. The current procedures bring about a variety of measures and solutions, but the society still continues to change and several challenges persist in dealing with the sharing of responsibilities and the development of participative management, within a new kind of governance, and in dealing with tools and scientific approaches such as quantitative approaches.

INTRODUCTION

Over the recent decades, most countries have experienced an expansion of their urban areas. The complexity of the technological and human systems has also increased considerably. This concentration of urbanisation and level of complexity has largely contributed to the increase of the global vulnerability of societies, albeit the occasional individual vulnerabilities may have reduced as a result of progress made in science. Despite the advances made in risk prevention, the society seems to be paradoxically less prepared to face disasters and alleviate their adverse effects. It is noteworthy that the approaches developed have not managed to successfully reduce the impact of natural hazards, and that a better knowledge of the phenomena involved is required to mitigate the increasing losses. This is due to the fact that risk management has remained, for too long, being concentrated on the strict analysis of the physical processes and favoured technical solutions and structural
measures rather than other more qualitative and more global solutions. It too often focuses on the short term and on the management of the crisis and dismisses the local know-how. From now on, risk management policies should adopt an integrated approach, involving all the stakeholders on the basis of a full diagnosis of the areas involved, far beyond the problems of natural risks alone, and also on the basis of the political ambition for land development, and of applying research findings and good practice. This would need to rely on strong and efficient communication amongst the practitioners.

Before the introduction of statutory power such as the publication of the ZERMOS maps (“Zones Exposées à des Risques de Mouvements du Sol et du sous-sol”, depicting zones exposed to ground and underground movements), landslide risk management is practised since the 1980s within the general policy of natural risks management.

Landslide risks induced by human activities such as earthworks, embankments or mining works are considered through other procedures: earthworks and embankments are a matter of classical engineering whereas mining works are managed within PPRm – i.e. mining working risk prevention plans. The distinction between the three categories is not always simple; ancient quarries are managed within natural risks procedures when they are considered as having been abandoned and when the owner or the operator is not known. If the design of an embankment is implemented within engineering input, the management for land-use planning is often considered through PPRN – i.e. natural risks prevention plans.

The need for a good theoretical knowledge of the mechanisms that drive ground instabilities, which is required for designing protection works or for stabilizing punctual phenomena, is taken for granted for decades. Technological improvements in the fields of data transmission, computing, data processing, etc. allowed major advances to be made in terms of risk management upon a much wider horizon.

However, do the current procedures relating to land-use planning fit with the present demands of the society in a moving institutional, social and economical context? Are the tools and models efficient enough to help in decision-making regarding operational risk management? What are now the challenges, the key problems the scientists have to solve so that decision-makers can give balanced answers, so that societal and environmental evolutions can be anticipated, such that financial investments can be optimized? Should natural risk management policy be driven within a merchandising approach based on cost-effectiveness, such as cost-benefit analysis? Should institutional evolutions, globalization of the world and increase of society individualism lead to an evolution of the responsibility in terms of natural risk management?

DEVELOPMENT OF SLOPE AREAS: AN OPPORTUNITY, AN ASSET, A RISK
At all times, man has been confronted with slopes and instabilities which were imposed upon him. Thanks to the power of technology and tools, man had thought that he could be freed from the constraints imposed by such an environment.

Man’s belief in the power of science and technology made him forget that the natural environment is fragile, that the natural, human and built environments are interdependent, that any action on one of the environments is liable to generate consequences on the other two. Any action that shifts the natural environment from its state of precarious balance must be compensated in return; the stronger and more brutal the action is, a greater imbalance will
result, and more violent consequences could be the outcome. The biosphere as a whole behaves to some extent according to the same principle of action and reaction as for the endocrinal system.

As nature is fragile, it has a considerable destroying capacity. Urban civilizations tend to forget about this through ignorance, arrogance, distrust, or quite simply due to the fact that they are plunged more and more in the virtual reality of the games. It is not only a question of one’s lack of education, but rather of a new type of relationship with nature. Our scientists and technical experts, brilliant as they may be, forget sometimes that if there are differences between their modelling and the reality of nature, the fault does not fall upon nature itself. Nature is never wrong. It sometimes brutally reminds us of this through the occurrence of catastrophic events.

But what is more, nature is also in perpetual evolution, and the local states of balance should not be divorced from the global modifications on the scale of the planet earth; thus, the temporal and spatial frames of reference do not remain constant. If man were able to understand, model and try to control the local systems, such is not the case with the global systems whereby he is only a spectator. We do not fully understand all the mechanisms of the global evolutions. The genius of some should not hide the fact that man only plays more of a role of a sorcerer’s apprentice with respect to the natural environment than as the enlightened manager. General relativity and the quantum theory do nothing but raise one small corner of the veil, which hides the integrated mechanisms of our planet as well as our ignorance.

The slope represents a particular context, which is exacerbated by the fact that the reactions can be very different from those observed on flat areas; they are sometimes amplified, attenuated, shifted or deformed.

Man cannot live on a slope. Its upright position and its psyche require both the flat areas and verticality, except when he sleeps. Just like “Dahu-bird”, the man of the slopes does not exist! Studies have shown that beyond some degrees, any slope can pose problems in terms of the space reference and balance. However, that does not mean that all ground must be flat. The slope can be considered as a whole, like a succession of planar levels connected by elements of transfer. Consequently how to develop this space, and how to arrange it without being exposed to natural or human risks, are the key questions.

Man has always known how to control the slopes. Through agricultural terraces or the most complex architecture, the ingenuity has made it possible to benefit from a difficult environment, by dominating it or by adapting to it. Round the world, there are lots of nice examples where areas were conquered on the mountains, sometimes in a spectacular way (see Figure 1). Such is not always the case, however (see Figure 2)!
The city and constructions, another fundamental aspect of the human activities, were also installed on steep slopes. The form, the continuity of the built masses, the orientation, the imitation with the environment, constituted as many factors of success the plain does not cause. The examples, here either, do not miss (see Figures 3 and 4).
Except for very rare exceptions, many of the constructions carried out on slope areas during the last decades, seem like mistakes or, at the very least, banal works that are badly integrated into the site. This may reflect the existence of extreme difficulties posed by slopes. Such is surprising. The past offers to us a multiplicity of remarkable examples that were well managed, by using simple techniques, whereas we have now much more efficient technologies. Without speaking about “lost science”, at least one can advance the idea of forgotten precaution.

All these installations, be they in the rural or urban setting, can simply be summarized, in a simplified manner, as the realization of horizontal platforms. But if in the past they were well designed, and well integrated into their environment, such is not the case with the contemporary achievements. Too many cities invade slope areas with heterogeneous constructions with aspect that are generally regrettable. Apart from the negative visual impact, these installations often expose the human lives and the structures to slope instabilities, sometimes with dramatic consequences. Some of the changing conditions certainly include the following:

- The demographic growth obliges to urbanize more and more quickly,
- The evolution of the urban life generates an increase in the consumption of spaces, even with a constant population,
- Public financial means are increasingly difficult to come about,
- The invading development of the mobile car modifies deeply the land-use.

The development projects carried out on slope areas during the recent years too often seem banal achievements, badly integrated into the site, or even dangerous. The majority of these developments are done in sectors where the natural environment prevails, where the repercussions of a bad integration to the site are highly visible. This absence of integration, however, does not come about because of technical problems.

The characteristics of the natural environments must always be taken in consideration in the in a development project. That is even more true on slope areas where the natural environment constitutes at the same time a constraint and a potential to be realised. Regarding the existing techniques of construction, the slope areas do not pose a major
problem per se. On the contrary, one can exploit the wonderful range of possible solutions in these areas.

If “bad” examples are numerous, there are hopefully some recent cases where the urbanization has adapted to the constraints of the slope areas, to the needs, to the means and to the ways of life of the population concerned, and one is fully known to have benefited from it.

If the building owners and project managers build their programs with a minimum of thought and ambition, taking into account a certain number of precautions as well as use of good practices, the adjustment to slope areas will become an additional chance for the urbanization.

EVOLUTION OF THE SOCIETY: AN OFFER FOR PROTECTION WHICH SHOULD HAVE EVOLVED

Evolution of Living: Urban Concentration and Modification of Natural Environment
Strong changes of land occupation have occurred during the last decades leading to a level of higher risk and obliging the public power to bring an answer adapted in terms of landslide risk management.

Society is becoming more individualistic. This translates notably at the planning level into the development of the ‘property owner’, through private housing, with the quest for a better standard of living; slope areas are more and more “colonised” around towns, with the implementation of road networks and infrastructure. In a similar fashion, the development of out-lying areas is found in developing countries, for obviously different reasons, but with identical consequences with respect to the human modification of the environment, the alteration of the drainage and the problems of uncontrolled excavations.

Urban concentrations are more and more important with, yet again modification of the environment, and a change to the pre-existing equilibriums. Cases of urban extensions carried out within the framework of an integrated development and a preliminary impact analysis - notably concerning the risks - are few and far between; at best, selective geotechnical studies are undertaken.

This urbanisation of the environment also corresponds to a strong shift of activities and lifestyles, with the discontinuation of agricultural practices generally accompanied by a reduction in land maintenance. Water flow management is no longer ensured, whether naturally by vegetation cover or by the implementation of collecting devices for drainage and/or irrigation. Water flow and drainage methods both on and within the ground and their imbalances can occur at a variable frequency. It is important to note that the upgrading of communication routes can contribute to an important modification of the flows, and even more so if the rainwater drainage systems have not been constructed or maintained properly.

Urban civilization, while moving the man away from nature, removes the benchmarks which the oral or written tradition could give him with regard to the natural phenomena, on the natural environment and on the great harmony that governs them. The reflexes of maintenance of the territory are reduced, which also contribute to the imbalance of the environments.
Evolution of Social Demand for Protection: The End of the Welfare State

The actions of reduction of the risk are defined with regard to the social demand for protection, development, and quality of life. However, the needs and the reactions of the society have strongly evolved for about one century with respect to natural risks.

The urban populations no longer accept any damages, even for the simple disturbances generated by phenomena that are supposed to be random. Moreover, as soon as a catastrophe occurs, the responsibilities and even the culpabilities are required with an increasing demand for compensation. This evolution of the society, which appears in different ways according to civil and penal codes of the different countries, has important consequences on risk management, and in particular on the political decision-making. The principle of precaution tends to apply strictly and the conflicts in land-use multiply.

To face this evolution of social demand, the political power has tried to develop tools and procedures aimed at controlling the development of the territories in the exposed areas, to standardize the procedures of risk analysis, to manage the crises and to compensate the victims. Has the risk management become easier? Not so! The population rarely accepts the conflicts of land-use, the restrictions of freedom, and the impacts on land value. One finds here the limits of a system that is strongly regulated and subject to constraints.

In highly protected societies, the demand for protection can become unreasonable, in particular when the protection is given by the public power. The adequacy of protection measures, the cost/benefit analyses and the choices of the level of protection must not only develop, but should also be the subject of a national debate with the general public. Such debates cannot be limited any more to single exchanges between the technical experts and the risk managers.

The society wishes from now on to take part in the debate and to participate in the formulation of the prevention policies. If the public do not wish to or cannot get involved in the scientific debate, the technical experts are not necessarily given a free hand. Control is typically carried out a posteriori, often following a catastrophe, and the critics are then often brutal and irrational, without any concession for the scientists and the technical experts: “if one can go to the Moon, one should be able to avoid the catastrophes!” Only a rigorous, explained and shared approach would make it possible to engage in a balanced debate. The qualitative approaches and the absence of explicit criteria for the acceptable risk do not however facilitate the task of the scientists.

Evolution of Political Response: from Repair to Prevention Toward Governance and Resilience

As is the norm, natural phenomena have surpassed our expectations, and man has tried to protect himself against the outbursts of fury of the forces of nature. Disasters have always had an impact on the imagination and have not been forgotten thanks to witness accounts. For example, letters from Pline the Younger gave an account of the destruction of Pompeï by Vesuvius.

If the first examples of risk management are very old, the natural phenomena however were perceived for a long time like the manifestation of divine anger (until the end of the 18th century in Europe). But natural risk management evolved, and the earthquake of Lisbon, on 1 November 1755, represents a first major stage, on the one hand in the awakening that the
human activity is a component of the risk, and on the other hand in the social demand for protection. In France, as in many countries, the regulation followed the evolution of society and the social demand for protection; it never anticipated them.

The fatalism that took place in the past was replaced by the search for responsibility, as well as for culpability. Previously based on the principle of repair and protection, risk management today seeks to anticipate catastrophes on a relatively strict lawful basis. The current evolutions of the society show that risk management through constraints is not acceptable any more. One tends to favour resilience and sustainable development based on a widen governance, and to convince rather than to constrain.

**Repair**

Without going back to the divine punishment, the public first of all accepted the damages inflicted by the various natural phenomena including fatalities. The forces mobilized can exceed the technical possibilities, and the population has, first of all, with some success and some failure, sought to be established in the least dangerous sectors. Since catastrophes occurred, man repaired the damages and went on living, “enriched” by a painful experiment and trying to forget the events.

![Figure 5: Rock falls. Woodcut. 16th century. Grenoble (in “les risques naturels”, Besson 2005)](image)

**Protection**

With the economic development and the need for establishments close to the rivers to profit from the means of transport, the population has gradually encroached upon the more exposed zones. If, as in the Middle Ages, one does not wonder about the mechanisms of the phenomena, one starts to organize oneself to bring about protection. Thus, one built dams to protect oneself, and later, the public power set up the RTM (Services de Restauration des terrains de Montagne) to face ground movement among other phenomena. But when the catastrophes occurred, fatalities prevailed.

**Land-use Development: Planning and Prevention**

After the Second World War, in order to rebuild a weakened country, the public power introduced a prevention policy for the development of its territory. Anticipation and planning had become a need. The consideration of natural risks was carried out via the code of town
The R.111-2 article of the code of urban planning is thus written after modification by the decree n° 98-913 of October 12, 1998: “The building permit can be refused or be granted only subject to the observation of special regulations if constructions, by their situation or their dimensions, are likely to affect the health or to public safety. It is the same if projected constructions, by their location near other installations, their characteristics or their situation, are likely to affect the health or public safety.”

The R.111-3 article of the code of urban planning stipulates that: “The building permit can be refused or be granted only subject to special regulations if constructions are likely, because of their localization, to be exposed to serious harmful effects...”

The ND zones of the POS (master land-use planning document): Founded by the law of land orientation of December 30, 1967, and supplemented by the law of January 7, 1983, the land-use planning document (P.O.S.) constitutes the background document of the city planning. It defines the economic, social and architectural objectives of the commune and it fixes the applicable rules of urban planning on its territory. It comprises protected natural zones, the ND zones, which cannot be developed because of the natural risks.

**Prevention, Compensation and Information: Mutual Management of the Risks**

Following the destruction of the country cottage of the UCPA by an avalanche in 1970, and in the same year because of the mud flow of Plateau d’Assy, the public power set up in 1982 the plan of natural risks exposure (PER). This procedure coupled for the first time the three major components of the natural risk management, namely the prevention, the compensation and the information for the public. It enacted the great basic ideas which were going to control the public policy of prevention of the natural risks in France:

- the responsibility of the State and the mayor
- the mutual management of the risks and the solidarity of compensation
- the information for the citizen vis-a-vis the natural disasters

Compensation for the victims of natural disasters: In its article 1, the law of July 13, 1982 stipulates that: “Insurance policies, […] open right to the people to be guaranteed against the effects of the natural disasters on the goods being included in such contracts. Are regarded as the effects of the natural disasters within the meaning of the present law, the direct material damages having had due determining the abnormal intensity to a natural agent, when usual measures to take to prevent this damage could not prevent their occurrence or could not be taken, natural disaster situation being declared by interministerial decree.”

Thus, on the other hand of the guarantee which is based on solidarity offered by the system of insurance of the goods exposed to natural risks by the law of July 1982, the people were to implement certain measures of prevention. For this purpose, the State had to establish PER to determine the exposed zones as well as the techniques of prevention to be adopted.

However, their development encountered certain difficulties. Indeed, compared to the perimeter of the R. 111-3, the PER presented a difficulty: the obligation of setting in conformity the constructions established before with the approval of the PER. Moreover, its implementation was an administratively heavy procedure. The PER, not having answered the
expectations of the legislator, the law of February 2, 1995 relating to the reinforcement of the environmental protection replaced the PER with the Plan of Risk Prevention (PPR), which is simpler to apply.

Since the enactment of this law, the prevention policy did not cease to be reinforced, with a Welfare State being responsible for the protection of its fellow citizens. Thus, several major laws were elaborate, in most cases following natural disasters, whose majority were related to the floods, which are the majority phenomena in France, but of which the effects were applied to the unit of the natural risks, movements of ground being included/understood.

These laws concern:

- **The right for information**: The law of July 22, 1987, modified by the laws of July 30, 2003 and August 13, 2004 stipulates that: “The citizens have to be informed on the major risks to which they are exposed in certain zones of the territory and on the safeguard measures which relate to them. This right applies to the technological risks and the foreseeable natural risks…”

- **The reinforcement of the environmental protection**: this law, also called Barnier Law, currently represents the major element of the prevention policy of the natural risks in France, in the sense that it institutes the concept of preventive expropriation, with a specific fund managed by the State, and because the Plan of Risk Prevention (PPR) replaces the PER. In its title II the clauses relating to the prevention of the natural risks stipulate that:

> Article 11 [...] when a foreseeable risk of ground movement, torrential flow, avalanche or flood seriously threatens the human lives, the goods exposed at this risk can be expropriated by the State [...] if the means for safety and protection of the populations are more expensive than the allowances of expropriation.

> Article 13: It is created a special fund for the prevention of major natural risks charged to finance, within the limit of its resources, the allocated allowances [...]  

> Article 16: [...] The State defines and realises the plans of prevention for the foreseeable natural risks such as the floods, the ground movements, the avalanches, the fires of forest, the earthquakes, the volcanic eruptions, the storms or the cyclones. These plans have as needed:

> 1° to delimit the zones exposed to the risks [...] to prohibit there any type of construction, work, installation or exploitation [...], [or] to prescribe the conditions under which they must be carried out, used or exploited;

> 2° to delimit the zones which are not directly exposed to the risks but where constructions, works, installations or exploitations [...] could worsen risks or cause new ones and to define the measures of prohibition or prescriptions as envisaged with the 1° of this article;

> 3° to define the measures for prevention, protection and safety which must be taken, [...] by the public bodies [...] and by private citizens;
4° to define [...] measures relating to installation, the use or exploitation [...] that must be taken by the owners, or users.

The realization of the measures envisaged with the 3° and 4° of this article can be made compulsory according to the nature and of the intensity of the risk within five year, or less in case of urgency. [...] 

Imposed works of prevention [...] to be realized by the owners or the users can only be limited.

- The “solidarity and urban renewal law”: the documents of urban planning must integrate the prevention of the natural risks, the plan of risks prevention being annexed in the Local Plan of Urban Planning as a constraint. The law of December 13, 2000 affirms that: “The plan of territorial coherence, the local plans of urban planning and the communal plan determine the conditions making it possible to ensure: [...] A sparing and balanced use of spaces natural, urban, suburban and rural, the control of the needs for displacement and motor vehicle traffic, the safeguarding of the quality of the air, water, ground and underground, ecosystems, parks, natural environment, sites and natural or urban landscapes, the reduction of the sound harmful effects, the safeguard of the remarkable urban sets and built inheritance, the prevention of foreseeable natural risks, technological risk, pollution and harmful effects of any nature. ...”

- The information of Purchaser and Tenants (IAL): this law reinforces the right for information by obliging the salesman or the hirer out to inform the purchasers and the tenants of the natural risks to which the good is potentially exposed. The ordinance of June 8, 2005 institutes the right to information for the purchasers or the tenants: “The purchasers or tenants of goods located in zones covered by a plan of prevention of the technological risks or by a plan of prevention of the foreseeable natural risks, prescribed or approved, or in zones of seismicity defined by decree in Council of State, are informed by the salesman or the financial backer of the existence of the risks aimed by this plan or this decree.”

Governance and Resilience
Following the questioning of the traditional modes of management of public policies, and with the abandonment of the State “providence”, the civil society and the individuals wish more still to involve themselves in the political decision-making. The natural risk management through a legislative framework increasingly more importantly does not meet any more adhesion of the population. The civil company, with its complexity and its individualism growing, wishes to take part in the definition of fundamental balances between the ambitions of freedom and the needs for protection. From victims, the individuals want to become actors and want to take part in the orientations of public policies.

In this new governance, the civil society asks that “the actors of any nature and the public institutions join, shares their resources, their expertise, their capacities and their projects, and creates a new “coalition” for actions based on the shared responsibilities”. This interaction is made necessary by the fact that no actor, public or private have alone the capability and the knowledge to solve the problems of natural risks prevention.

Governance thus implies participation, negotiation and coordination.
A new concept appeared recently in the risk management, namely, that of resilience. This term taken from physics, where it represents the aptitude of a body to resist the pressures and to take again its initial structure, was used also in the 1990s by the American psychiatrists specialized in early childhood to describe a capacity of living, succeeding, or developing despite adversity. This term described, according to Mangham: “the capacity of the individual to face a difficulty or an important stress, in a way not only efficient, but likely to generate a better capacity to react later to a difficulty”. It was then popularized in France by Boris Cyrulnik, by considering that “Resilience” defines the capacity nevertheless to be developed, in environments which should have been dilapidating.”

It is now used in the natural risk management context to describe the capacity of the society to face the risks successfully, to re-establish the initial situation, and “to even rebound” higher. Resilience rests on the concepts of protection, training, stress, difficulty as well as on the possibilities of finding some help, and some support.

In a context where the public power tries to impose a more pragmatic management of the risks, and where the society is not very inclined to accept any recession, one understands how the concept of resilience can be of interest. The feedback experience constitutes an important tool, so that the damaging phenomenon can be presented, if not like a happy event, at least like an element making it possible to reinforce the society vis-à-vis the future threats of nature.

MILESTONE EVENTS WHICH LED TO THE EVOLUTION OF LANDSLIDE RISK MANAGEMENT

Fiz Cliff Collapse (Roman time)
The first known major ground movement incident in France was the Fiz cliff collapse (located above Servoz in Haute-Savoie). The landslide debris dammed up the Arve river, creating a lake, which the Romans had then to reduce it by excavating an underground gallery.

Mount Granier (1248)
One of the greatest historically known phenomena, which resulted in many victims, occurred during the Middle Ages (November 24, 1248). It involved the collapse of Mount Granier (Savoy), which led to 2000 to 5000 deaths and destroyed 5 villages. The contemporaries believed the event was a punishment from the sky, punishing the Duke of Savoy of the alliance who contracted in 1247 with Frederic Barberousse, emperor of Germany, and perished two years later. Following intense rainfall, Mount Granier would have stored great quantities of water in its low part. The ground mass started to slip carrying with it part of rock cliff. The energy released by this catastrophe evaporated part of the water contained in the material. It was conjectured that the vaporization along the slip surface carried the mobilized material on an air cushion, which may explain the runout of the deposit and the speed of the propagation. The material covered a surface of 20 km², that is about 8,5 km in length and 5,2 km in width, which amounted to some 150 million cubic metres in volume.

Fiz Cliff Collapse (1751)
On August 4, 1751, starting from cliff of Fiz, rocks detached, crushing 6 people, 90 cows and destroying 3 barns. Ten days later, on August 14, 22 millions cubic metres of material collapsed causing a ground motion that was felt down by people in Piedmont of Italy.
Fourvière Landslide (1930) and Aristide Brian Landslide (1932)
The town of Lyon witnessed a certain number of serious events related to slope instability which dominated the Rhone and the Saone. The most well known with the most number of fatalities occurred on the night of November 12, 1930 with the catastrophe at Fourvière, which killed 41 people, and in 1932 another landslide caused 30 fatalities in Aristide Briand.

Figure 6: Lyon, Saint-Jean – Fourvière landslide in 1930

Petit-Clamart Caving-in (1961)
“On June the 1st, 1961, an enormous underground rumbling, and a few minutes later 6 hectares of chalk broke down from 2 to 4 metres in the communes of Clamart and Issy-Les-Moulineaux. Six streets disappeared and the stadium of Issy Moulineaux was transformed into lunar landscape. One counted 21 dead, 45 wounded, more than 273 victims and 23 buildings destroyed. The causes of the catastrophe were never clearly identified. Some speak about a collapse due to the bad condition of the career working.”
Mud Flow of Plateau d’Assy (1970)
In the night of Thursday April 16, 1970, a strong mud flow resulting once again from the Fiz clives, destroyed a sanatorium located on the slope under the “Plateau d’Assy” (Haute-Savoie), killing 72 people including 56 children. This major event reinforced the concerns of the State as regards prevention of natural risks. Plans showing zones that are exposed to the movements of the ground and underground movements (i.e. the ZERMOS plans), and the promulgation, 12 years later, of a specific law dedicated to natural risk management, are the direct consequences of this milestone event.

More recently, in 1994, the landslide of La Salle-en-Beaumont (Isère) resulted in 4 victims and destroyed several dwellings.

Those events that remain in the memory of all as major phenomena, also illustrate the evolution of the society regarding natural risk management, the evolution of risk perception and the evolution of the society’s demand for protection. If the Mount Granier collapse is recognized as a major event with more than 2000 people dead, one needs “only” 72 deaths
for the mud flow of Plateau d’Assy and later 4 victims with La Salle-en-Beaumont landslide to make these events milestone events, which influenced the French regulation context.

**GENERAL PRINCIPLES OF NATURAL RISK MANAGEMENT IN FRANCE**

In France, natural risk management is divided into three parts:

- The prevention of the risks, under the responsibility of the ministry for Ecology and the durable development which has inter-ministerial responsibilities;
- The civil protection (intervention in case of crises and preparation of the corresponding planning), under the responsibility of the ministry for the interior, interior safety and local freedoms;
- The compensation which is managed through insurance, for which the ministry for the Economy, finances and the industry plays a dominating part.

The French specificity made it possible to develop prevention based on a “jacobine” (centralized) approach. Repairs are also managed.

The laws of 1982, 1987, then of 1995 and finally of 2000 constitute the principal reinforcement of natural risks in France, the risk prevention plan (PPR) constituting a major element of natural risk management. This one associates the regulation of land-use, the information on the populations and the compensation in case of a catastrophe. However, it remains to be founded on a technical approach in respect of the prevention of the natural risks, integrating in an insufficient way the local ambitions for development of the local decision-makers.

These laws fall under the French republican tradition of national solidarity vis-a-vis natural disasters, and rest on the general principles that are presented hereafter.

**Protection of Human Life**

The protection of the human lives represents the first objective of the prevention policy of natural risks in France, before any economic considerations, even if for quite some time new evolutions have tried to bring the protection policy back to a more reasonable level. The State “providence”, the principles of solidarity and mutual management of the risks, as well as the absence of definition of criteria of risk acceptability and quantitative risk analysis, culminated in the population losing a sense of responsibility, becoming a society that has increasing requirements in terms of protection.

The preventive expropriation defined under the Barnier law (law of 1995) represented the first significant evolution in an “unbounded” protection system, an analysis of “profitability” being realized whenever a serious and eminent risk threatens human lives. Consequently, a preventive expropriation could be committed. The rise and modulation of the premiums of insurance, defined by the State, reinforced this evolution, without however calling into question the basis of the prevention policy of natural risks regarding protection of human lives.

**Principle of Solidarity**

As a pillar of the French Republic, the principle of solidarity was reinforced since 1982 with the enactment of various laws relating to the prevention of risks. National solidarity with respect to natural disasters is a guiding principle for the safety of people; thus the State is the guaranteeing party, and for this reason it is responsible for the prevention as for civil protection.
Whatever the level of exposure, that the person is or is not located in an exposed zone, the premium of insurance regarding natural risks will be identical. In other words, there is no individual modulation of the insurance according to the level of exposure. Whether one is or is not exposed to the risk, each citizen pays an “extra premium” on the basis of the ensured good, and the rate of extra premium is fixed by the State. In any case, the insurances can index the amount or the rate of the extra premium according to the level of risk, and an insurance can end a contract under the pretext of a natural risk.

The Mutual Management of the Risk
The mutual management of the risks is the corollary of solidarity. The funds collected by the insurance companies for the natural risks and gathered in a specific fund called Cat-Nat fund, are managed by an inter-ministerial commission. Guaranteed by the State, this fund aims to compensate the victims for the natural disasters since the state of natural disaster will have been recognized by the State through an inter-ministerial decree.

The Link “Prevention – Compensation – Information”
The legislator wished to associate the prevention of risks with the information of the citizen and the compensation for the victims of catastrophes under the same law, in order to reinforce the principle of solidarity.

The population is at the same time victim, actor and criterion of reference of the risk, and its multiple expectations, sometimes contradictory or even irrational, in the majority of the cases, are based on sector analyses. Thus, the demand for protection was continually increasing. But it must be satisfied, generally by the public power, the Welfare State, without encroaching upon individual freedom, and without any decrease of the purchasing power. The population wants all of these at the same time, protection and freedom, and without additional expenditure.

The State Responsibility
The State is responsible for natural risk management in France, for the prevention, the organization of the rescue, and the compensation. The implementation of this policy, managed at the central level by various ministries, is under the responsibility of the prefect with his various operational services.

This responsibility is shared at the local level with the mayor, first representative of the State as regards natural risk management. The mayor ensures that this responsibility is under his policing powers, in order to guarantee the safety of its citizens. In this case, it takes care that the documents of urban planning integrate correctly with the prevention of the risks, and that no building permit is delivered for developments in zones with a high risk.

The citizen must also ensure his part of the responsibility by respecting the zones with prohibition to build, and by implementing the regulations formulated in the Risk Prevention Plan. It has for that purpose a maximum amount of time of 5 years. Nevertheless, these prescriptions should not induce costs higher than 10% of the monetary value of the exposed goods.

The Components of Landslide Risk Prevention and their Application
The seven components of the prevention of the natural risks such as they are exposed by the public power are as follows:
Knowledge about the risk
- Monitoring – alarm
- Information
- Consideration of the risks in land-use planning (PPR)
- Works
- Preparedness and management of crisis
- Feedback experience

**Knowledge**

Spreading information regarding natural risks is under the responsibility of the State. Initially, the State has a national database on landslides created at the end of the 1990s, by joining together the databases developed within the public organizations and para-public LCPC, BRGM and RTM services. This database is accessible via Internet (www.bdmvt.net) and gives basic information. A database also exists on the abandoned underground cavities “except mines” (www.bdcavite.net). These two databases are managed by the BRGM.

In addition to those databases, the improvement of knowledge is also achieved within the risk prevention plans when technical experts produce the hazards maps, which are carried out at 1 to 25,000 or at 1 to 10,000 in urban sectors with high stakes.

**Monitoring and Alarm**

With the development of new communication and information technologies (NTIC), and the emergence of new tools of instrumentation, monitoring, in particular remote monitoring, became major elements for landslide risk management. In the mountainous areas, the topographic constraints associated with strong environments and generally limited financial resources often make remote monitoring and early alarm systems the only effective means to reduce the risk. Experience shows that in case of damage to the infrastructures of transportation, the interruption has major economic repercussions. Thus, beyond the protection of the people, the protection of the goods represents a stake that is always important, and for this reason, remote monitoring constitutes today a relatively reliable means and economically viable tool to prevent or limit the losses through a more effective management of the risk.

In addition, the concept of economic profitability of the instrumentation remains always delicate to “sell” to “decision-makers” since the surfaces to be covered are important, and since the phenomena present slow kinematics. Moreover, the strategy to be adopted differs according to whether one uses the monitoring for infrastructures of transportation with a discontinuous human presence in space and in time, or if it is used “to protect” inhabited places.

In France, the monitoring of ground movements is generally under the responsibility of the commune, the owners and the managers of networks; some large phenomena being managed by the State, as is the case for the Rockslide “Les Ruines de Séchilienne” (see Figure 12) and the landslide of La Clapière, but at this time no strategy of systematic monitoring of the territory was implemented.

The new tools such as the airborne lasers and the terrestrial lasers could change this strategy taking into account their resolution, their space coverage and the speed of implementation and treatment.
The systems of detection or monitoring aim to alert the persons in charge of the occurrence or the imminence of a dangerous phenomenon, but thresholds remain still difficult to define.

Figure 10: National data bases on landslides and underground cavities (except mines)

Figure 11: Example of phenomenon map and landslide hazard map
The objectives of preventive information are to inform the population about the risks to which it is exposed in various places of life and various activities. It thus contributes to prepare the citizens to adopt a responsible behaviour. Several specific laws were promulgated to organize what is considered by the public power as one of the basis of the risk management, in particular the law of 1987. They defined the contents and the methods of the compulsory communication. This information rests thus on documents prepared by the State (the DDRM and the DCS) and by the commune (the DICRIM and the PSC), on information brought also by the salesman or the hirer out within the IAL law of June 8, 2005, and on the definition of codes for the management of crisis. It is supplemented by information given by the civil society, by the use of new technologies, with the publications on line, in particular via the site of the ministry for Ecology, and Sustainable (www.prim.net).

The DDRM: this is a document of awareness, illustrated by risks maps, gathering basic information on the natural and technological risks at the department scale and fixing the communal priorities. Established by the services of the prefect for the departmental actors of the risk, his objective is triple: to mobilize the local decision-makers and their partners on the risks in their department and their communes, in order to incite them to develop information, to be the reference document for the realization of the synthetic communal document (DCS), and to nourish and enrich through information all the actions implemented in the department. It must be updated every 5 years.

The DCS: The synthetic communal document for major risks aims to inform and make the population of the commune aware of the natural and technological risks and of the safety measures to be implemented. It is carried out by the services of the State and is notified by the “préfet” through a decree to the mayor, so that one can work out the DICRIM. The DCS comprises certain elements related to the commune (information, maps) extracted from the DDRM with possible additions (presentation of specificities of the commune).
cartographic documents of the DCS are not compulsory, neither for land-use nor as regards insurance contracts.

The DICRIM: the communal document of information on the major risks is obligatory for nearly 18,000 communes in France; to date a little more than 7,000 have such a document. Worked out by the commune, it checks the safety measures, in particular those taken under the policing powers of the mayor. The DICRIM is carried out starting from the DCS, and it comprises synthetic sheets dedicated to the public.

The PCS: The Communal Plan of Safety starts again from the DICRIM, and supplements it by the presentation of the means of coordination to be mobilized and implemented on the commune, obligatory information (risks maps, particular action plan) are given by the State services: DRIRE, DIREN, DDE.
Figure 15: DICRIM - Marseille

Figure 16: PCS – Nancy

Figure 17: Codes implemented by civil protection
**Consideration of Risk in Land Development**

Taking the risks into account in the planning rests to date essentially on the Risk Prevention Plan (PPR). PPR determines the zones exposed to natural dangers and defines the conditions of land-use, from zones where any development is prohibited to the construction under conditions. Non-exposed areas are not constrained, and only the national rules of construction apply to them (anti-seismic, anti-cyclonic rules).

In accordance with the code of urban planning (L.121-1 article), “the plan of territorial coherence, the local plans of urban planning and the communal plans determine the conditions making it possible to ensure: […] 3° […] prevention of the foreseeable natural risks, the technological risks, pollution and the harmful effects of any nature”.

Since the PPR is approved by the prefect, it is annexed in the Local Plan of Urban Planning as a constraint of public utility. It has a retroactive effect on the goods and existing activities.

If the commune has no PPR, the mayor has to take into account the prevention of the risks in the projects of land development; he uses if necessary the R111-2 article of the code of urban planning. This information rests on the existing studies, maps and data which are gathered by the State. The delivery of the building permits is under the responsibility of the mayor, the State ensuring a control of legality.

**Protective Works**

The works implemented to reduce the risks aim to:

- To reduce the hazard
- To guarantee the protection of the existing facilities
- To reduce the vulnerability of the exposed facilities
The works can be made compulsory with PPR, but in this case, their amount cannot exceed 10% of the value of the exposed facilities. The owners have 5 years to be in conformity with the regulations of the PPR. Two major points are to be noted:

- Regarding the general philosophy of the State, protective works are only aimed at protecting the existing facilities. They do not have the role to open new areas to urbanization. Thus, it is not possible to build behind protective works. This position is derived on the one hand from the fact that the prevention policy of the risks assessment is based on a qualitative approach, and on the other hand from the fact that the protective works cannot generally guarantee an effective protection in the long term. Based on the feedback experience, this is true with the risk of flood and with rupture of embankments. The catastrophe of Gondo in Switzerland with the rupture of a protecting wall and the destruction of several houses is a good example of this uncertainty. In the strongly constrained territories, as is the case in mountainous areas, specific exemptions can be considered if maintenance is guaranteed.

- Since a PPR is prescribed (for a commune) and approved (for a private person), and if measures are made compulsory by the PPR, the State can subsidize the works to a total value of 30%. The subsidies are allotted on a case-by-case basis following an analysis of the situation.

**Preparedness to Crisis Management**

The preparation for crisis management belongs to the prerogatives of the State and the mayor. This preparation has, on the one hand, to check the elements at risks on the territory, and to identify those which are strategic elements for the organization of the rescue, and on the other hand, to inventories and present risk reduction measures which are implemented, as well as the available means.

Those elements of information must be presented in the Communal Plan of Safety prepared under the responsibility of the mayor.

**Feedback Experience**

The last component of the prevention, the feedback experience, aims at reinforcing the society after the occurrence of a damaging event. In terms of resilience, the objectives are to learn from the past in order to improve the global answer of the society and thus to reduce its vulnerability.

The feedback experience also aims to improve the scientific knowledge of the phenomena, and to provide information on the damage levels and on the associated costs. Feedback experience is organized either through national missions or from local missions. Each year, a synthesis is written by the Ministry for Ecology, and Sustainable Development.

**Other French Specificities in Landslide Risk Management**

In addition to those inherent in the institutional and legal context, and in addition to the points already mentioned in the preceding sections, certain French specificities must be mentioned:

- There is not, to date, any quantified strategy for risk reduction, neither on a national scale, nor on a local scale;
- Landslide risk assessment stills remains qualitative, and this highlight the point above;
- No systematic monitoring of ground movements has been implemented in France; only a few large phenomena such as “Les Ruines of Séchilienne” (Isère) and the Landslide of
“La Clapière” (Saint-Etienne-de-Tinée - Alpes-Maritimes) are monitored continuously; the new tools for spatialized remote monitoring such as laser airborne could make it possible to set up different strategies;

- Methods for assessing and mapping natural risks were defined on the basis of floods, especially the scales of investigation, the qualitative approach, and the scale of mapping. The principles which prevailed for floods are however not always relevant for the other phenomena, particularly for ground movements. Those methods show their limits and some evolutions are in progress, at least in practice if not in the form of official directives.

**NATURAL RISK PREVENTION PLAN (PPR)**

**Objectives, Procedure and Contents**

PPR aims to analyse the risks on a given territory, to define from this analysis the areas at risk, to recommend the development on those zones that are free from risks. It also aims to prescribe regulations as regards urban planning, and to prohibit any construction or activity where the risk is too high.

**A State Responsibility**

PPR is financed and carried out by the State, on the scale of 1:10 000 in urban areas and 1:25 000 in rural areas. At the end of 2005, the 5000 most exposed “communes” (France counts 36,000 “communes”) were equipped with a PR.

**A Four-step Procedure**

- The prescription: the PPR is prescribed by the prefect (it is carried out under the control of the services of the State);
- The consultations of the town councils and the population, and public inquiry;
- Approval: approved by the prefect PR becomes a constraint for public utility, and is appended to the master urban planning document;
- Mandatory information: the decree of approval is opposable only at the end of the formalities of mandatory information (in administrative acts of the State in the department, in two local newspapers, posting in a town hall and making it available to the public).

**Contents of the PPR**

Unified document of prevention, realized with pragmatism from existing knowledge and on the basis of a qualitative study, it is composed of three elements:

- A report which presents:
  - the reasons of the prescription of PPR
  - known existing phenomena (inventory and cartography)
  - hazard (modelling and mapping)
  - the stakes (qualitative inventory and cartography)
  - objectives for the prevention of the risks
  - choice for mandatory risk mapping and compulsory measures
- A mandatory risk map which comprises two zones:
  - the red zone where no development, building or activity is allowed
  - the blue zone which makes it possible to have buildings and activities if some measures of prevention are undertaken
- A regulation
The Regulation
It comprises:

- Prohibitions and prescriptions: for new constructions, the main principle is to prohibit development in the zones most at risk. In the sectors where new constructions are allowed, they are subjected to compulsory prescriptions. The regulation for urban planning must be clear, realistic and balanced with the stakes;
- General measures for prevention, protection and safety: they include measures to be taken by the private person, and collective measures under the responsibility of the public power;
- Measures applicable to existing buildings and activities: they relate to land development, land-use or exploitation. They concern the existing buildings and all types of activities. The inhabitants of the zones covered by a PPR must however preserve the possibility of conducting a normal life or activities if they are compatible with the required objectives of safety. Measures for prevention, protection and safety, can be made compulsory within a maximum of 5 years with possible execution by the State if they were not carried out in due time.

Hazard Assessment and Mapping
Landslide hazard assessment as well as mapping are carried out in accordance with a methodology defined by the ministry for Ecology, and Sustainable Development; this methodology is presented within specific guidelines which rest on:

- The inventory and description of the existing ground movements and indices of instability as identified in the field;
- Analysis of the typology and the extent of the phenomena identifying the factors of predisposition (topography, lithology, etc.) and of the triggering factors (earthquake, rainfall, anthropogenic activities, etc.)
- The explicit formulation of the criteria characterizing the conditions of instability for the landslide hazard taken as the reference (topography, geometry of the failure surface, lithology, soil moisture or seismic ground motion);
- Hazard assessment based on indicators that allow the characterization of the intensity of the phenomenon. The indicators are of two types:
  - Direct indicators: mobilized volume
  - Indirect indicators:
    - The «gravity» (see Table 1); defined taking into consideration the expected human damages;
    - The “aggressiveness” (see Table 2): defined from the type of expected damages;
    - The “quest for potential protection” - DPP (see Table 3): defined from the expected measures needed to guarantee protection.

The correspondences between the various indicators are defined in Table 4.
Table 1: Example of ground movement gravity scale with four level re. human live threat (LCPC-CFGI 2000)

<table>
<thead>
<tr>
<th>Gravity</th>
<th>Human live threat</th>
<th>Examples of ground movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very low</td>
<td>No fatalities or rare</td>
<td>Swelling soils, mining subsidence, slow landslides (&lt; 1m/h)</td>
</tr>
<tr>
<td>Medium</td>
<td>Isolated fatality</td>
<td>Landslide with high run out speed (few dam/h), isolated rock falls</td>
</tr>
<tr>
<td>High</td>
<td>Few victims</td>
<td>Rock falls, mud flows</td>
</tr>
<tr>
<td>Major</td>
<td>Few tens of victims</td>
<td>Collapse, major mud flows, generalized caving in</td>
</tr>
</tbody>
</table>

Table 2: Example of ground movement aggressiveness scale regarding buildings (LCPC-CFGI 2000)

<table>
<thead>
<tr>
<th>Agressiveness</th>
<th>Expected types of damages</th>
<th>Example of phenomena</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Fabric of the building scarcely affected</td>
<td>Subsidence of low range, rock falls, shallow landslide</td>
</tr>
<tr>
<td>Medium</td>
<td>Fabric of the building affected with possible repair (e.g. Moderate failures)</td>
<td>Subsidence of high range, punctual collapse of small size, rock fall, landslide of limited extend</td>
</tr>
<tr>
<td>High</td>
<td>Fabric of the building severely affected (or collapse), impossible to use the building anymore; expensive repairing or not possible</td>
<td>Collapse, large landslides, large underground caving in</td>
</tr>
</tbody>
</table>

Table 3: Example of a 4 levels ground movement intensity scale based on “Potential Demand for Protection” (PDP) (in LCPC-CFGI 2000)

<table>
<thead>
<tr>
<th>PDP</th>
<th>Level of necessary preventive measures</th>
<th>Example of preventive measures</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>Affordable by a single owner (cost less than 10% of the value of an individual dwelling)</td>
<td>Purge of few unstable blocks in a cliff, reinforcement of a cavity with few concrete pillars</td>
</tr>
<tr>
<td>Medium</td>
<td>Counter measures affordable by a small group of owners (collective building, small housing estate)</td>
<td>Earth fence against rock falls, underground cavity fill in, small drainage system</td>
</tr>
<tr>
<td>High</td>
<td>Specific counter measures that concern a large area or of high cost</td>
<td>Large landslide stabilization, stabilization of a large cliff</td>
</tr>
<tr>
<td>Mjor</td>
<td>No counter measure that are technically or financially acceptable</td>
<td>Exceptional phenomena (i.e. Séchilienne -Isère, or la Clapière - Alpes-Maritimes) of tens millions cubic metres</td>
</tr>
</tbody>
</table>
Table 4: Assessment of rock fall or rockslide intensity after different definitions based on mobilised volume (LCPC-CFGI 2000)

<table>
<thead>
<tr>
<th>Mobilized Volume (V)</th>
<th>Gravity (Table 1)</th>
<th>Agressiveness (Table 2)</th>
<th>PDP (Table 3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V &lt; 1 dm³</td>
<td>Very low to medium</td>
<td>Very low to low</td>
<td>Low</td>
</tr>
<tr>
<td>1 dm³ &lt; V &lt; 100 dm³</td>
<td>Medium</td>
<td>Low to medium</td>
<td>Low</td>
</tr>
<tr>
<td>0.1 m³ &lt; V &lt; 1 m³</td>
<td>Medium to high</td>
<td>Medium</td>
<td>Medium</td>
</tr>
<tr>
<td>1 m³ &lt; V &lt; 1 000 m³</td>
<td>High to major</td>
<td>Medium to high</td>
<td>Medium</td>
</tr>
<tr>
<td>1 000 m³ &lt; V &lt; 100 000 m³</td>
<td>Major</td>
<td>High</td>
<td>High</td>
</tr>
<tr>
<td>100 000 m³ &lt; V</td>
<td>Major</td>
<td>High</td>
<td>High to medium</td>
</tr>
</tbody>
</table>

In the methodological guidelines on the realization of landslide PPR, it is recommended that the risk should be assessed from existing data, based on simplified qualitative approaches. The objectives are to ensure coherence with the whole natural phenomena and in particular with the flood which was used as reference for the development of the national methodology. Thus, no test, no complementary investigation and no model is required. Taking into account the constraints which can generate PPR on land development or on properties, such approaches are penalizing.

In practice, the technical experts use more and more modelling, the costs of PPR not being necessarily higher.

![Figure 19: Landslide hazard map – commune of Clefs (Source: RTM 74 in Ministère de l’Aménagement du Territoire et de l’Environnement 1999)](image-url)

**Inventory, Analysis and Vulnerability of Stakes**

The cartography of the stakes, combined with hazard is the basis of the risk calculations and in consequence of the definition of the mandatory zoning. Hazard mapping is based on the identification of the human, socio-economic and environmental stakes, highlighting those which are sensitive elements (worsening or reducing the risks), or strategic ones (taking part...
in the rescue or the decision-making). Thus urbanized areas are identified, protected or not by works, where the constraints will be lower not to constrain too much the development, if protection measure are implemented.

Moreover, the ambitions for development by the municipality on its territory are realised through its master document for urban planning, in order to identify sectors with conflicts between development and protection.

Structural and non-structural elements which represent stakes and which are vulnerable are also identified: population, buildings, administration, as well as various networks.

Finally, environmental stakes that need to be protected are identified.

**Mandatory Zoning**

The PPR is not an urban planning document; it is a public constraint which is applicable to all, State, communes and citizens. It is retroactive and applies to the existing facilities and activities. Zoning map is presented at 1 to 10,000 scale.

Derived from hazard maps and stakes maps, according to the general methodology as adopted by the French ministry of Ecology and Sustainable Development, the mandatory zoning aims to regulate the land use, whether they involve constructions or activities. Thus, measures are of three types:

- Measures leading to prohibit any development, construction or activity (depending on the specific conditions of the site);
- Measures leading to specific constraints (high, medium or low), in particular building under certain conditions (depending on the specific conditions of the site);
- Measures leading to normal constraints which apply to the whole territory and which generally relate to construction codes.

![Stakes map - commune of Terre-de-Bas (Les Saintes – Guadeloupe)](image)

Figure 20: Stakes map - commune of Terre-de-Bas (Les Saintes – Guadeloupe) – Source URBATER : (a) urbanized areas, (b) urban planning map,
For those zones where all developments are prohibited, measures that make it possible to better control the risks and to improve safety of the people already present, are to be taken, objectives being to not increase the population and the facilities within the exposed areas. Some developments, works or exploitations can nevertheless be allowed there in order to allow the occupants to have a normal life or activity, if safety can be guaranteed.

Areas where buildings are allowed under conditions are zones where the potential phenomena do not directly threaten human lives; this can be because of the low level of the hazard intensity, or because the probability of occurrence is low. It is however recommended to take particular measures in order to limit the risks for the pre-existing people and facilities:

- In the zones with high specific constraints, it is possible to build or develop after a preliminary study has been carried out under the responsibility of the public power (commune), and which will aim to reduce the vulnerability and to control the stakes;
- The zones with medium specific constraints can be developed subject to individual and/or collective measures aimed at reducing the risk;
- The zones with low specific constraints can be developed subject to individual measures aimed at reducing the risk;

The zones with normal constraints can be developed taking into account only national codes dealing with earthquakes and/or cyclones.

The colours which are used was defined in the general guide for PPR. Zoning is based on the three colours: red, blue and white, even if some evolutions tend to identify new specific areas (areas of high importance for the development) with specific colours. The general rules are as follows:

**Table 5: General rules for the pattern colours of the mandatory risk map**

<table>
<thead>
<tr>
<th>Zone</th>
<th>Level of contraint</th>
<th>Nature des prescriptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red</td>
<td>Prohibition</td>
<td>No de elopment</td>
</tr>
<tr>
<td>Dark blue</td>
<td>High specific constraint</td>
<td>Preliminary global study for land development under the control of a public authority</td>
</tr>
<tr>
<td>Blue *</td>
<td>Medium specific constraint</td>
<td>Individual to collective prescription /measure</td>
</tr>
<tr>
<td>Light blue</td>
<td>Low specific constraint</td>
<td>Individual prescription /measure</td>
</tr>
<tr>
<td>White</td>
<td>Normal constraint</td>
<td>National building codes</td>
</tr>
</tbody>
</table>

Note: * this zone is specific to flooding
Figure 21: A mandatory multi-risks zoning – Commune of Deshaies (Guadeloupe)

The Limits of the PPR
Appended to the master plan for land-use planning, the PPR (zoning and regulations) represents a strong constraint in terms of urban development and town planning, in particular for the attribution of the building permits. The constraints are of three types:

- scientific: at a scale of 1:25 000, the qualitative studies based on existing knowledge and of the limited budgets for study, often result in working out, under the “umbrella” of the principle of precaution, too pessimistic zonings;
- Strategic: applying to existing facilities, the impacts to the value of land and building properties are significant;
- Methodological: the stakes mapping do not take into count well enough the ambitions of the local developments, even if a dialogue exists with the commune; this can lead to major conflicts between the State and the commune regarding the balance to be struck between development and protection.

The New Tools
Despite the different phases for exchanging of information, which are clearly defined in the procedure, PPR remains perceived by the mayors as an interference of the State in the local development, and also as a constraint “blocking” the development. PPR proposes solutions for risk reduction and not prospects for development. This new approach aiming at developing an integrated methodology taking into account risk in the land-use and in the development of the territory is needed; the Ministry for Ecology and Sustainable Development is developing such an approach with the local projects for natural risk prevention.

To articulate, to confront the question of the risks with the forthcoming projects for development, such is the objective of the implementation of a local project for natural risk prevention. Answering this question forces the “communes” concerned to raise the true questions of development, and to identify the main stakes for their future development; it
then requires to conceive the suitable answers through dialogue with the parties of the development themselves. Such a project is implemented within a five-step approach:

- The risk assessment,
- The evaluation of the vulnerability of the elements at risk, and the severity of the potential damage,
- The definition of the strategy for the local development of the territory,
- The establishment of a local project,
- The implementation of the project.

Since the community conforms to this approach, the State is likely to finance a part of it, to a total value of 50% for the studies and 30% for works.

Furthermore, the State aims to promote an urban-planning-integrating-risk-approach, which would make it possible in certain cases not to implement PPR.

**CHALLENGES**

**Scientific Challenges**

The processes which lead to instabilities of slope are, in theory, relatively well known; this can concern stability models, behaviour laws, models of propagation, etc. The models which were developed are particularly powerful. Uncertainties remain however, in particular on the boundary conditions, on the mechanical characteristics of the materials, on the accurate geometry of the different layers, etc. but these are generally not explained nor presented at all.

The developments of the personal computers during the last decades make it possible from now on to have capacities of calculation largely higher than what the operational users expect. Thus, neither the comprehension of the phenomena, nor the capacities of calculation represent up to date the genuine limitations of deterministic analyses, and of quantitative risk assessment. The limits are elsewhere, and it is necessary to make the methodologies evolve, to implement new approaches, to integrate new disciplines, in particular social sciences, to improve the global knowledge, and to make scientific 'bolts' blow up. Among these, one can mention the following:

**The Non-stationary Behaviour of Time Series due to the Forthcoming Climate Changes and Evolution of the Territory**

This point is particularly important in France where the natural environment dominates and is still largely protected, but where the sub-urbanisation has strongly developed for a few decades.

The use of deterministic models is limited most of the time to engineering studies on specific and active phenomena. Its generalization to land-use planning is only at its infancy. In many cases, the evaluation of the risks rests on empirical approaches and laws, on the basis of time series and feedback experience, by using the principle of causality, that is to say that the same causes generate the same consequences.

The principle of causality as applied to strongly modified “natural” processes, if not even controlled by the man, is not necessarily valid. The robustness and the simplicity of this
principle rest on a strict similarity of the factors of causality intervening directly on the process. It is not because the rainfall series remains constant that the consequences will be similar; the modification of the natural environment, in particular in suburban areas, and the absence of maintenance of the territories, sometimes radically, modify the modes of propagation and concentration of water in the ground. Transfer functions between rainfall in a given place, and the pore water pressure in another place are modified, sometimes strongly. The same causes will thus not generate the same consequences.

The climate changes reinforce this drift, even if the modifications imposed by the man regarding intensity and temporal variability as well, are much higher than those induced by nature itself.

Few studies were carried out to quantify the incidences of the modification of natural environment, of the abandonment of agriculture, even with relatively simple approaches based on Manning-Strickler coefficient.

The influence of the anthropogenic activities on triggering slope instabilities is major, even for deep-seated landslides; it is however not immediately obvious and the modifications can take several years or even few decades. Thus, beyond his impact on the stakes (urban concentrations in areas at risks), man can increase in a very significant way the risk while intervening in a more or less direct way on the trigger of the phenomena, and on their frequency.

How should one consequently integrate in a realistic way these modifications in the time series in order to have stable scientific models? In addition, does the concept of annual probability generally used to characterize the risk remain relevant taking into consideration fast and important modifications imposed by man?

**To Fit the Realisation of the PPR with the Specificities of Ground Movements (scale and 3D modelling)**

The methodology developed for the implementation of the landslides risk prevention plan mainly rests on flooding. Regarding the scales of investigation (1/10 000 or even 1/25 000) and the type of approach (qualitative or “simple”), the philosophy of the PPR rises from the flood risk.

The consequences induced by ground movements on the society in terms of risk zoning are important; the energies to be counterbalanced may exceed the technical possibilities, in particular at acceptable cost. Consequently, medium risks may lead to prohibition of development, even if some evolutions can be observed; the consequences are strongly negative, as well as for the communes whose development can be stopped, as for the citizens penalized regarding their properties. Any approach contributing to raise in an excessive way the zones where development is prohibited, under the “umbrella” of the principle of precaution, because of lack of knowledge or inaccuracy, cannot be accepted.

Simple qualitative approaches at a scale of 1/10 000 are not relevant for ground movements. It is necessary to more precisely identify the exposed zones; this is why the use of modelling tends to be often essential. 3D models are increasingly accessible; it is recommended to encourage their use, to generalize them, and to develop operational tools for the practitioners.
Developing Frequency – Magnitude Law to Improve Quantitative Approaches

Few Frequency-Magnitude laws were implemented in the field of ground movements. They are particularly important for implementing quantitative approaches for assessing risk. It is necessary that the public power can begin work in this field, on the basis of a national inventory of existing ground movements, and puts the results at the disposal of the engineers.

Systematic Campaigns of Multi-temporal High Resolution Aerial Photos (National Database)

The development of new space technologies for investigating the territory offers interesting prospects for identification, follow-up and modelling of ground movements. The existing data are also very few and often accessible at prohibitive prices. It would be particularly convenient if the public power would invest in systematic campaigns of multi-temporal aerial photographs, and if the data could be made available for the professional to work out more relevant risk map, to develop better models of propagation and to set up basic monitoring on a national scale.

Such a program would make it possible to propose a quantified strategy for landslide risk reduction in France, on the basis of financial programming.

To Develop the Use of the Lasers in the Field of the Monitoring to Optimize the Financial Investments (Broader Space Cover) and Not to Focus Only on Large Phenomena

The contributions of lasermetry are manifest, and the results obtained on some ground movements show this. If the costs of the materials remain still important, they do not represent necessarily the critical point in the use of this technology. The modules of treatment are still confidential, and few commercial tools are now accessible to the practitioners.

To Model the Impact of the Urbanization on the Flows and the Trigger of Ground Movements (Mud Flows in Particular)

As a complement to what was presented for time series, modelling the impact of the anthropogenic actions on a territories basis regarding the water run-off and the trigger of the ground movements would represent a need in terms of management of the territory. It is necessary to work out a research program aiming at the following:

- To show the influence of anthropogenic actions on the modification of the risks known as natural;
- To quantify the modifications due to the evolution of the ground cover, which will make it possible thereafter to carry out prospective modelling (scenario) taking into account the ambitions for development shown in the documents of urban planning;
- To inform the general public regarding the incidences of the individual actions on circulations of water. This teaching approach will make it possible to better render in a comprehensible way the measures recommended in the PPR and the PLU on the management of surface water flows;
- To develop methods for assessing the vulnerability, as well as cost / benefit analysis of risk reduction measures (“profitability” analysis).

Studies dedicated to landslide vulnerability are insufficient. However this component is necessary to carry out quantitative evaluations of the risk. The implementation of international program aiming at developing damage functions for different types of exposed good, and at identifying suitable reinforcement measures or protection for different types of
pathology, would make it possible to formulate realistic regulations in terms of risk reduction, in particular in the ALARP area.

It would also be necessary to define the levels of investment which one is prepared to accept, on the one hand to leave the zone of the intolerable risk, and on the other hand to reduce the risk in the ALARP zone without adopting a commercial approach of the risk.

**Challenges in Dealing with Land Development**

Beyond the scientific contingencies, the limits are also of a social, economic and political nature. The challenges are multiple. Some of these are discussed in the following:

*To better take into account the political ambitions of development in the PPR*

PPR comprises procedures that can strongly constrain the territories; if PPR does not create the risk (it simply reveals the existence and distribution of the risk), its evaluation largely remains to be based on the hazards involved and is thus largely driven by technical considerations. The political ambitions of development of the communes are still insufficiently taken into account in practice, and the perception as a negative document remains the major feeling among the local authorities as well as among the general public.

As the decision-makers and urban planners must integrate very upstream in regard to the prevention of risks in their projects, it is fundamental that the technical experts of the risk should integrate as soon as possible in their approaches to the future development projects. Such an approach will not only make it possible to better define the level of investigation to be dedicated to a given territory, but it will also make it possible to put forward more suitable measures for risk mitigation.

*Risk-based urban planning: putting less constrains with the PPR*

To limit the number of PPR, to reduce the constraint induced by the PPR, to improve the responsibility of the general public and the local decision-makers, such are some of the key ambitions driven by the risk-based urban planning approaches.

Implementing win/win risk management strategies represents the best way for guaranteeing perennial solutions for risk mitigation. Urban planning, architecture, landscape development are some examples of the many opportunities for integrating risk prevention in the various projects for territorial development. To integrate the constraint due to the risk in the projects without using an external mandatory procedure, as with PPR for example, and to let urban planners themselves take risk into account in their own projects, often one can make it possible to bring more relevant and less expensive solutions. Various examples already exist as with the management of rainwater by the techniques of soft hydraulics.

The integration of the slope in the projects also represents interesting opportunities to implement relevant solutions to reduce the risks and to give additional value to the projects themselves.

The existing experiments should be promoted within methodological guidelines, if possible within international guidelines.
To Develop Landslide Quantitative Risk Assessment Approaches to Rank the Response of Non-development in the Exposed Sectors

Last but not the least, the development of landslide quantitative risk assessment methods represents today a major field of interest for improving landslide risk management. They make it possible to rank the responses in terms of development in exposed areas, by avoiding the too simple binary approach “development or no-development”, in particular behind earth fences, by introducing lightly secured developments based on acceptable risk criteria.

CONCLUSIONS

Landslide risk prevention and landslide risk management can be reduced to specific compulsory procedures, such as the PPR for example, even though since the 1980s. France has been implementing a very comprehensive legislative framework dedicated to risk.

A global approach has to be adopted within a logic of actors; the responsibilities, the expectations and the needs are sometimes conflicting, if not antagonistic. Such global approaches must rely on a close dialogue and comprehensive communication: a good rule is that which is known, understood and accepted.

In France, natural risk management in which landslide risk management is included, is first of all under the responsibility of the State. It comes from the French republican tradition of solidarity and mutual management to face natural disasters. But this responsibility is shared: by the local authorities on the one hand, in particular by the mayors, and the population on the other hand. The time of the Welfare State is over, and the behaviour as well as the practices must evolve to implement new answers that are better adapted to the complexity of the problems and to the complexity and the evolution of the society.

The tools and the procedures implemented by the public power in France made it possible to reach an important level of protection, even though it is neither quantified nor evaluated. However, the society continues to evolve with increasing expectations as well. The citizens want to become the actors of their own protection and to take part, with the communes and the State, in the decision-making. Natural risk management must from now on, be considered within a new governance, a widened governance. That requires an evolution of the tools and procedures, but moreover of the relationships between the various actors in charge of the development and the protection of the territory. To convince more than to constrain, to share more than to oppose, such are the new paradigms which must relay the compulsory constraints to lead to a risk-based urban-planning methodology through partnership and synergy of the means.

The urban evolution, the modification of the natural environment, and to a lesser extent to date, the climate changes, induce immediate or long term modifications which can have major effects on natural risks. The tools and the models must evolve, together with the concepts and the practices. International collaboration should help improving knowledge transfer and implementing suitable guidelines.

REFERENCES

LANDSLIDE RISK MANAGEMENT:
COUNTRY REPORT – AUSTRALIA, 2007

Andrew Leventhal
Australian Geomechanics Society and GHD Geotechnics, Sydney, Australia

Abstract: The tools available to assist practitioners and regulators in Landslide Risk Management have advanced in Australia this year (2007) through the publication of a suite of guidelines developed under the auspices of the Australian Geomechanics Society. The introduction of the concepts of Landslide Risk Management into government regulation is continuing.

INTRODUCTION
The application of Landslide Risk Management in Australia has advanced in several significant steps over the last two and a half decades and is now embedded in regulation in a number of Local Government Areas (these being important as areas within which residential development is susceptible to landslides) and a State Government instrumentality in New South Wales (a state on the eastern seaboard of the Australia).

This paper provides an overview of the status of Landslide Risk Management in Australia as a contribution to the International Forum on Landslide Disaster Management held in Hong Kong from 10th to 12th December, 2007.

BACKGROUND ACTIVITIES PRIOR TO 2007
A sub-committee of the Australian Geomechanics Society (Walker et al. 1985) introduced the concept of risk into hillside residential development. It is fair to say that this was the introduction of the concept of risk into the residential development process (through the building approval process) within Australia. Pleasingly, it was rapidly accepted and adopted by Local Government Areas within critical metropolitan areas of Sydney and Melbourne.

Fell (1995) presented a keynote address on the style, mechanics and distribution of landslides within the southeastern Australia. Excluding mining activity, he concluded that: most landslides in Australia are in soil and weathered rock reflecting the deeply weathered profile over much of the country; most landsliding is restricted to a few geological environments; the vast majority of sliding is reactivation of existing natural instability; many soils are fissured, and shear strengths between residual and fully softened are appropriate; many sedimentary rock and tertiary sediment slides occur where low residual strength soils and rocks are present, and it is likely that sliding has occurred due to the presence of bedding plane shear; much instability is rainfall related, and landslide activity has increased through clearing of vegetation; and whilst large costs had been spent on infrastructure in the late 1980’s and early 1990’s in New South Wales (NSW), Fell’s view was that greatest costs were in the large areas of potential residential land effectively sterilized by being zoned as subject to unacceptable risk of landslide.

It is noted also that the wide spread instability that affected significant lengths of the Main Northern Rail Line and the over 100 sites on the South Coast Rail Line in NSW from 1988 to
1990 were principally a result of an La Nina-driven extended period of rainfall.

In 1997, a landslide occurred in the ski resort village of Thredbo that demolished two accommodation lodges and killed 18 people. The fatalities were the subject of a Coronerial Inquiry (Hand 2000).

In 2000, the Australian Geomechanics Society published a technical paper on landslide risk management concepts and guidelines (AGS 2000). Several years earlier, it had been recognized that the 1985 advice had been outdated through improvements within the practice of risk assessment and risk management, both within Australia and internationally, and the re-constituted sub-committee was working towards this as updated advice to geotechnical practitioners and to regulators. AGS (2000) superseded the 1985 advice. Within his determination of the Thredbo Inquiry, Coroner Hand (2000) recommended “that the Building Code of Australia and any local code dealing with planning, development and building approval procedures, be reviewed and, if necessary, amended to include directions which require relevant consent authorities to take into account and to consider the application of proper hillside building practices and geotechnical considerations when assessing and planning urban communities in hillside environments”. He further recommended that AGS (2000) “be taken into account in undertaking this exercise”.

Most domestic assessments conducted in accordance with the principles and guidelines within AGS (2000) are performed as qualitative assessments for risk to property, with a quantitative (or perhaps more correctly, semi-quantitative) assessment of risk to life. Some Local Government Councils operate with acceptable risk levels of “Moderate” for property and 1E-5 per annum for risk to life within the domestic development setting, whilst others set acceptable risk levels of “Low” or “Very Low” for property and 1E-6 per annum for risk to life. AGS (2007c) and AGS (2007d) recommend, for new slopes or for new development of Importance Level 2 or for existing landslides, that risk values of “Low” for property and 1E-5 per annum for risk to life be adopted as “tolerable” (which implies an order of magnitude higher risk than an “acceptable” level, this being a trade-off between the risks, the benefits of development and the cost of risk mitigation).

AGS (2000) has been adopted within planning instruments by Local Government Areas such as: Pittwater Council (in the northern beaches area of metropolitan Sydney), Wollongong City Council (in the Illawarra area on the south coast of NSW), Shire of Yarra Ranges (outer metropolitan Melbourne in Victoria) and Colac-Otway (in rural Victoria); and State Government instrumentalities such as the NSW Department of Planning for Kosciuszko National Park (which covers the alpine ski resorts of New South Wales – including Thredbo) and the Victorian Alpine Resorts.

A discussion of the status of adoption of Landslide Risk Management around the Australian Governments is provided within the appendix of Leventhal & Kotze (2007). Therein, it is noted that:

(a) **Nationally.** For residential development, the Building Code of Australia requires every site to be classified in accordance with AS2870, which is an Australian Code that deals with the identification and management of reactive clays. AS2870 permits classification of a site as Class P for circumstances not covered by identified reactive clay scenarios. Such Class P situations, whilst perhaps mainly intended for sites with a significant presence of fill or soft soils, could also include landslide hazard and/or landslide risk.
This, however, is perhaps relatively tenuous, and to the authors’ knowledge has not been tested.

The guideline for landslide hazards (ABCB 2006), developed by AGS for the Australian Building Codes Board, is a companion document to the Building Code of Australia and has introduced the concept of risk management for landslide issues. Currently, this guideline is an advisory (rather than mandatory) document.

(b) The regulations for each State and Territory are quiet varied, few recognize the issue of Landslide Risk Management and there is intermittent reference only to AGS (2000).

Quantitative studies have been conducted for particular major infrastructure projects, such as the Bethungra Spiral on the Main Southern Rail Line between Sydney and Melbourne (Moon et al. 1996) and for Lawrence Hargrave Drive (Wilson et al. 2005). The scale of the projects has permitted undertaking of these studies.

In 1998, a major storm event lead to 140 separate landslide events (fortunately with no attributed fatalities) throughout the Illawarra Region on the South Coast of NSW, i.e. within the Local Government Area of Wollongong City Council. The actions within this emergency led to recognition through an award from Emergency Management Australia. An outcome of the actions undertaken during the event led to the development of a Landslide Action Plan (Wollongong City Council 1999). A copy of which is attached herein as Appendix B. It would be appropriate for this Action Plan now to be updated through reference to Appendix C of AGS (2007c), specifically in regard to the risk matrix presented within the plan.

PROGRESS IN 2007
The major activity to be reported for 2007 is the publication of the suite of guidelines and commentaries published in the March 2007 (Vol. 42, No. 1) issue of Australian Geomechanics (see Table 1). The development of these guidelines and their commentaries was funded under the 2004-2005 National Disaster Mitigation Program funding round. The application was sponsored by the Sydney Coastal Councils Group.

Copies of the guidelines and commentaries are available for download from the Australian Geomechanics Society’s website: www.australiangeomechanics.org [from the home page use the link “Download the Landslide Risk Management documents”, and then download from AGS (2007)]. Note that copies of AGS (2000) are also downloadable from the same page.

The Landslide Zoning Guideline provides guidance in the methods of landslide zoning to government regulators (officers of local government and state government instrumentalities) and geotechnical practitioners. Such characterisation will provide input to the planning process in areas of landslide hazard. The associated Commentary provides background to the guideline. This guideline and its commentary are very similar in structure and content to the guideline that was developed in parallel by JTC-1.

The Practice Note Guideline and Commentary provides guidance both to practitioners in the performance of project specific landslide risk assessment and management, and also to government officers in interpretation of the reports they receive. The Practice Note can be used as an external reference document for legislative requirements and supersedes the recognised industry "standard" on Landslide Risk Management in Australia – AGS (2000).
AGS (2000) remains as a complementary commentary and reference document. The Practice Note is a means to provide uniformity in the quality of assessment and reporting and so will promote confidence in the planning and risk management process in regard to landslide hazards.

Table 1: List of guidelines and commentaries in *Australian Geomechanics* Vol.42, No.1

<table>
<thead>
<tr>
<th>Guideline Title</th>
<th>Abbreviated Title</th>
<th>Reference</th>
<th>Intended Users</th>
</tr>
</thead>
</table>

The Practice Note provides:

(i) a revised risk to property matrix to address shortcomings identified in usage;
(ii) recommendation for the adoption of tolerable risk criteria for risk to life;
(iii) the introduction of Importance Levels and linked tolerable risk criteria for risk to property;
(iv) the introduction of a suite of model sign-off forms, linked to recommendations from risk assessments, to improve the linkages between assessment, design and construction. This provides a management tool in the Landslide Risk Management process;
(v) further explanation of the risk equation and method of calculation, together with further examples and references and
(vi) guidance on the contents of a Landslide Risk Management report.

The Australian GeoGuides for slope management and maintenance provide owners, occupiers and therefore the public in the broader sense with guidance on management and maintenance of properties subject to landslide hazard.
The suite of guidelines and the Australian GeoGuides benefit the general community and Local Government regulators through achieving safer, more sustainable communities in relation to their exposure to landslide risk. The guidelines also reduce risk to the community through improved planning and slope management practices – key requisites of the National Disaster Mitigation Program funding. These guidelines link with the risk management practices presented in AGS (2000) and as enhanced by the Practice Note and the Building Code of Australia Guideline (ABCB 2006).

This suite of guidelines contributes significantly to completion of the Landslide Risk Management framework for Australia described in Leventhal (2007) and Leventhal et al. (2007). Appendix A contains a project sheet that briefly explains the project and its outcomes. A diagram depicting the Landslide Risk Management framework is included within the third page of the project sheet.

As part of an undertaking to distribute the outcomes of the project, CD-ROMS containing copies of the guidelines and commentaries were distributed to each Local Government Council throughout Australia.

It is noted that the use of Importance Levels, as defined in the Building Code of Australia, has enabled a move from strictly residential domestic development to a wider range of structures, e.g. from buildings which need to withstand a rapid onset natural emergency to farmyard structures. An explanation of Importance Levels and a copy of the discussion contained within the Practice Note Commentary (AGS 2007d) is provided in Appendix C.

Other Activities in Australia
The Department of Mineral Resources, Tasmania, is one of the few, if not the only, State Government instrumentalities in Australia involved in landslide susceptibility mapping on a regional scale (1:25,000). (The mapping program of Wollongong City is a Local Government undertaking – see below). Mineral Resources Tasmania, in its undertaking to provide an assessment of landslide susceptibility of major urban areas, has continued its mapping programme with the publishing of susceptibility mapping for Launceston, which complements earlier work around Hobart. Deterministic GIS modeling techniques were employed to produce predictive susceptibility maps. Mazengarb (2007) reported the status of this work, which aligns with the guideline (AGS 2007a). The outcomes are being used by Councils to identify the need for detailed assessment in response to development applications.

Mapping of landslide susceptibility and hazard mapping has continued within the Wollongong city boundaries through a combination of support from the Wollongong City Council (local government) and university sponsored research. This undertaking previously was also supported by State Government road and rail transport instrumentalities. Flentje et al. (2007) reports trialing an extension of the program into the broader Sydney Basin through the use of GIS methods.

Landslide hazard and susceptibility mapping was completed for Local Government land use planning within Pittwater Council’s area of responsibility in the northern beaches area of metropolitan Sydney in 2007 (Leventhal & Kotze 2007). Landslide likelihood is one of the most important input parameters to Landslide Risk Analysis, and research into this in the Pittwater area was also reported this year (MacGregor et al. 2007). This was supported by the work on rainfall analysis (Walker 2007) and on recorded rockfall frequency (Kotze 2007).
The Australian Government’s instrumentality, Geoscience Australia, has undertaken an assessment of natural hazards throughout Australia. The study (Geoscience Australia 2007) was conducted under the aegis of the Council of Australian Governments and addresses tropical cyclones, flood, severe storm, bushfire, earthquake, tsunami and landslides. As a consequence of the development of the practice of landslide risk management within Australia, a significant contribution was made to the landslide chapter by members of the AGS Landslide Taskforce. The landslide chapter deals with: hazard identification; costs of landslides; potential influence of climate change; roles and responsibilities; and discusses information gaps. The information gaps identified include the development of landslide inventories (a matter being addressed by Geoscience Australia through its Landslide Inventory Interoperability Project), support for regional susceptibility mapping; and support of the need for systematic and standardized landslide risk assessments throughout the nation (as is now possible through AGS (2007c) for example). This project promotes the AGS (2007) suite to government at all levels throughout Australia.

FUTURE WORK
Future tasks include:
- Modifications to regulations within the existing legislation is required to incorporate the AGS (2007) suite. This will initially involve Pittwater Council, Wollongong City Council, Kosciuszko National Park, Victorian Alpine resorts erosion management plan (under which landslide risk management is covered) and the Shire of Yarra Ranges and Shire of Colac-Otway in Victoria.
- Formulation of Development Control Plan - format for the performance of Landslide Risk Management within the building approval process, particularly for it to be suited to NSW Planning standard template which is under government consideration.
- Introduction of an Australia-wide / state-wide proforma for conducting Landslide Risk Management for the advantage of both regulators and practitioners, and hence for the advantage of the general public as well.

CONCLUSIONS
Whilst not expected to be fully comprehensive, this report provides the flavour of the state of Landslide Risk Management within Australia.

The major initiative completed in 2007 is the suite of guidelines and commentaries under the suite (AGS 2007), following over two years of development. This generation of risk management tools will promote the understanding and application of Landslide Risk Management in Australia.

REFERENCES


ACKNOWLEDGEMENTS
The development of the AGS (2007) suite of guidelines was developed through the efforts of the members of the Landslide Taskforce and, in particular, the three Working Groups and specifically their convenors and principal authors, Robin Fell, Bruce Walker and Tony Phillips. Their contribution has been rewarded through receipt of the Warren Medal for 2007 from the Civil College of the Institution of Engineers Australia. The input of the members of each Working Group is acknowledged.

The suite of guidelines was developed with funding support from the Australian National Disaster Mitigation Program, which is gratefully acknowledged. The support of the Sydney Coastal Council Group as sponsor for the development of the AGS (2007) suite of guidelines is also gratefully acknowledged.

APPENDICES
Appendix C – Extract from Building Code of Australia – Importance Levels
APPENDIX A
NATIONAL DISASTER MANAGEMENT PROGRAMME LANDSLIDE RISK
MANAGEMENT PROJECT INFORMATION SHEET, MAY 2007

NATIONAL DISASTER MITIGATION PROGRAM
PROJECT REPORT
July 2007

Project Outcomes:
- Guideline for Landslide zoning for land use planning.
- Guideline for Slope Management & Maintenance.
- Landslide Risk Management Practice Note.

Historical Background: In recognition of the challenge between development pressures and landslide hazard, in the year 2000 the Australian Geomechanics Society (AGS) published a benchmark technical paper “Landslide Risk Management Concepts and Guidelines” (AGS, 2000) – which significantly updated an earlier 1995 guideline. The purpose of AGS (2000) was to: establish uniform terminology; define a general framework; provide guidance on risk analysis methods; and provide information on acceptable and tolerable risks for loss of life. It represents a continued recognition by AGS of the pragmatic benefits of incorporation of the concept of risk in the assessment of potential landslides, particularly in planning and management situations. AGS (2000) was the culmination of a seven-year review that was in response to increased appreciation in Australia and internationally of the benefit of a risk management approach to landslide assessment and management. It was recommended in the report of the Coroner’s Inquiry into the 1997 Thredbo landslide that AGS (2000) be taken into account - through directions in the Building Code of Australia and local codes dealing with planning, development and building approval procedures - when assessing and planning urban communities in hillside environments.

Whilst AGS (2000) presented concepts and guidelines to assist practitioners, there remained a need to provide supplemental information to further assist practitioners, to assist regulators and to provide advice to the broader Australian population. This was recognised by the SCCG and in turn by the NSW and Commonwealth Governments through the National Disaster Mitigation Program.

The Project: The Sydney Coastal Councils Group (SCCG) and the Australian Geomechanics Society (AGS) have jointly conducted significant projects on the management of Landslide Risk. The projects were assisted with funding under the National Disaster Mitigation Program (NDMP). The projects were developed under the aegis of the AGS Landslide Taskforce.

The projects are intimately related to management of risk associated with landslides in all parts of Australia, covering sloping terrain generally and with applicability to the coastal and near-coastal environment.

The project involved the development of 2 guidelines, 2 commentaries and a suite of GeoGuides. These provide assistance variably to regulators, practitioners and owners and occupiers of property and land potentially subject to landslide hazards.

Description: There is a natural synergy between the guidelines, with common development and interlinking between them, as a result of commonality of purpose and their relevance to the development of a national framework for landslide risk management (see diagram below).

The Landslide Zoning guideline covers landslide susceptibility, hazard and risk zoning for land use planning. It provides guidance to government regulators (officers of local government and state government instrumentalities) and geotechnical practitioners in the methods of Landslide Hazard Zoning. Such characterisation provides input to the land use planning process in areas of landslide hazard.

The Slope Management guideline – known as the Australian GeoGuides for Slope Management and Maintenance - provides owners and occupiers, and therefore the public in the broader sense, with guidance on management and maintenance of properties subject to landslide hazard. These two guidelines are important contributions to the management of landslide hazard at both ends of the process – initial identification of landslide hazard in the planning process, and management of properties prone to landslide hazard by the end-user.
The guidelines benefit the general community (Australian GeoGuides) and Local Government regulators (Landslide Zoning) through achieving safer, more sustainable communities in relation to their exposure to landslide risk, and reduce risk to the community through improved planning and slope management practices. These guidelines link with the risk management practices presented in AGS (2000) and the recently published Building Code of Australia (BCA) Guideline, and will provide long-term natural disaster mitigation benefits to housing and infrastructure.

The LRM Practice Note provides guidance to practitioners in the performance of project specific landslide risk assessment and management, and also to government officers in interpretation of the reports they receive. The Practice Note is suited to be an external reference document for legislative requirements. The Practice Note supersedes the recognised industry "standard" on LRM in Australia – AGS (2000).

The Practice Note has application under the particular requirements of NSW SEPP 73 Kosciusko Alpine Resorts and the Pitwater Geotechnical Policy. The Practice Note also has application under the requirements of NSW SEPP 71 Coastal Protection. Further, it is envisaged that the Practice Note will have application nation-wide – for example under the Victorian Alpine Resorts Planning Scheme. This will provide uniformity in the quality of assessment and reporting, and so promote confidence in the planning and risk management process in regard to landslide hazards.

For the first time, there is formal guidance for the geotechnical profession or regulators as to acceptable minimum levels of investigation and the extent of reporting required for documentation to support Development Applications in areas prone to landslide. There is some guidance for projects within the Pitwater LGA (which is within Sydney's northern beaches) under the Interim Pitwater Policy (2003), though this policy will benefit from an ability to reference a credible external source as to acceptable practice. Furthermore, the desirability of formal guidance on acceptable levels of investigation has wider relevance, both state and nation-wide.

The guidance provided by the Practice Note is of a technical nature and is mainly for the geotechnical practitioner. This includes guidance on appropriate methods and techniques, and tolerable levels of risk. Currently, an acceptable level of risk to life for the individual most-at-risk is reasonably well identified, though perhaps conservatively so. As a result of a number of issues, there has been no similar guidance for risk to property. This is now provided in the Practice Note.

The Practice Note provides guidance for the production of landslide risk assessment and management for development in areas prone to landslide hazard. This could involve development upon slopes and cliff lines prone to instability, but also is applicable to development which can be impacted by instability that occurs uphill, adjacent or downhill of a subject site.

Project Quality: The SCCG established an External Observer Group to provide a national perspective to activities as well as a means for national implementation of outcomes from the project. The members of the External Observer Group include managers of commonwealth and state government departments and local government areas responsible for coastal processes throughout the nation.

A peer review process was implemented by AGS. A nation-wide peer review was performed for the technical portions of these guidelines. The Geotechnical Expert Panel of the SCCG and the External Observer Group each also were responsible for review.

The output from the studies is nationally endorsed guidelines for Landslide Risk Management.

**How does this all fit together?** As the LRM research, volunteer commitments and funding have come together over the last decade, a nation-wide framework for landslide risk management has become feasible. It is fair to say that the pieces of the jigsaw are now close to fitting together. The figure shows how the pieces of the process are coming together. The essence of this is:

1. The technical basis is provided by AGS (2000).
2. The Building Code of Australia provides an overarching legislative requirement, with a guideline on the applicability of LRM published in 2008 by the Australian Building Codes Board (ABC).
3. Implementation of universal and uniform policies at commonwealth, state and local government levels would be beneficial to all parties.
4. Hazard zoning guidelines for legislators is provided by the AGS Landslide Taskforce guideline for Landslide Zoning (AGS 2007a) – see also its commentary, AGS (2007b).
5. Recently published landslide likelihood research provides a degree of "base case" data as a starting point for semi-quantitative or quantitative assessments.
7. Slope management principles are provided for the owner and occupier through the Australian GeoGuides for Slope Management and Maintenance (AGS 2007e).
8. Technical competence of practitioners can be demonstrated through the specific area of practice within the National Professional Engineers Register (NPER).
The flow diagram shows the inter-relationship between each of those elements, and the outcome of a systematic and defensible landslide risk management process throughout Australia.

Outcomes: The guidelines were published in the March 2007 edition of the AGS journal, Australian Geomechanics (Vol 42, No 1), and are downloadable from the home page of the AGS website: www.australiangeomechanics.org

Want more information? Contact the SCCG:
email: info@sydneycoastalcouncils.com.au
or by phone on: (02) 9246 7791.
APPENDIX B
WOLLONGONG CITY COUNCIL, LANDSLIDE ACTION PLAN (1999)

BACKGROUND EVENT THAT LED TO DEVELOPMENT OF LANDSLIDE ACTION PLAN
A landslide emergency situation occurred in the Wollongong City Council area in August 98. It can be summarised as follows:

The Illawarra received an extended period of rain over July 98 to mid August. On 17 August the rainfall peaked with an extreme intensity short duration storm event which concentrated particularly between the suburbs of Mt Kembla and Bulli. A state of emergency was declared.

On 19 August, a large scale landslide occurred on Mt Kembla such that at 1AM, the emergency authority evacuated around 100 residents.

Many reports of landslides were being received such that, with a forecast of on-going rainfall, the emergency authority determined to form a geotechnical team to coordinate the assessment of the reported landslides.

The Geotechnical Team comprised Peter Tobin (Geotechnical Manager, Wollongong City Council) with Peter Stone (GHD-LongMac, now GHD Geotechnics) and Dr Phil Flentje (University of Wollongong). The team was assisted by geotechnical staff representing the Roads and Traffic Authority, Rail Services Authority and National Parks & Wildlife Service. The team visited sites as they were reported, initially with the assistance of NSW Police PolAir helicopter, and subsequently at ground level by vehicle and on foot. The team established a risk assessment process based on matrices developed in response to the Thredbo Landslide for initial ranking of the landslide inventory. From the initial visual assessment of risk to both person and property for each site, the reported landslides could then be prioritised and ranked for attention from the emergency response crews. Evacuations were recommended where the initial risk to persons was assessed as very high.

In the weeks following the storm, over 140 landslides were reported of which half had no previous history of slope instability. A database of landslide events was established, using MicroSoft Access, to manage the collection and storage of data, and reporting of the status of individual landslides.

As a result of the response to the emergency event, Wollongong City Council were recognized within the Emergency Management Australia Safer Community Awards 1998-99 as New South Wales and then National winners, in the category of Post-Disaster - Local Government Stream.

LANDSLIDE ACTION PLAN
A copy of the Wollongong City Council’s Landslide Action Plan is attached.
Wollongong City Council

Landslide Action Plan

A Sub-Plan
to the
City of Wollongong
Local Disaster Plan
Authorisation

This Plan is authorised in accordance with the provision of the State Emergency and Rescue Management Act 1989 and is a Sub-Plan to the City of Wollongong DISPLAN.

Recommended

________________________
Functional Co-ordinator – Geotechnical Services Sub-Committee

Dated: ______________________

Approved

________________________
Chairperson
Wollongong Local Emergency Management Committee

Dated: ______________________
Amendment Certificate

Suggested amendments or additions to the contents of this plan are to be forwarded in writing to:

The Chairperson
Wollongong Local Emergency Management Committee
Locked Bag 8821
SOUTH COAST MAIL CENTRE NSW 2521

Amendments promulgated are to be certified in the following table when entered.

<table>
<thead>
<tr>
<th>Amendment List Number</th>
<th>Date</th>
<th>Amendment Entered By</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15/7/99</td>
<td>P Tobin (GS)</td>
<td>20/8/99</td>
</tr>
</tbody>
</table>
List of Abbreviations

The following abbreviations are used in this plan:

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>DISPLAN</td>
<td>Disaster Plan</td>
</tr>
<tr>
<td>EOC</td>
<td>Emergency Operations Centre</td>
</tr>
<tr>
<td>GT</td>
<td>Geotechnical Team</td>
</tr>
<tr>
<td>LEMO</td>
<td>Local Emergency Management Officer</td>
</tr>
<tr>
<td>LEOCON</td>
<td>Local Emergency Operations Controller</td>
</tr>
<tr>
<td>NPWS</td>
<td>National Parks and Wildlife Services</td>
</tr>
<tr>
<td>RSA</td>
<td>Railway Services Australia</td>
</tr>
<tr>
<td>RTA</td>
<td>Roads and Traffic Authority</td>
</tr>
<tr>
<td>SES</td>
<td>State Emergency Service</td>
</tr>
<tr>
<td>SRA</td>
<td>State Rail Authority</td>
</tr>
<tr>
<td>UOW</td>
<td>University of Wollongong</td>
</tr>
<tr>
<td>WCC</td>
<td>Wollongong City Council</td>
</tr>
</tbody>
</table>
Table of Contents

INTRODUCTION .........................................................................................................................1
CONCLUSIONS..........................................................................................................................6

SECTION 1  INTRODUCTION .................................................................................................1
1.1 GENERAL ............................................................................................................................1
1.2 PURPOSE ..............................................................................................................................1
1.3 DEFINITIONS .......................................................................................................................1
1.4 ROLES AND RESPONSIBILITIES .......................................................................................1
   1.4.1 Local Emergency Operations Controller (LEOCON) ......................................................1
   1.4.2 Geotechnical Team (GT) ...............................................................................................2
1.5 TEST AND REVIEW PROCESS ..........................................................................................2

SECTION 2  RESPONSE ............................................................................................................3
2.1 ACTIVATION ........................................................................................................................3
2.2 COMMUNICATIONS ..........................................................................................................3
2.3 WORK METHODS ...............................................................................................................3
   2.3.1 Immediate Action Plan .................................................................................................3
   2.3.2 Risk Assessment ...........................................................................................................5
   2.3.3 Inspections ..................................................................................................................5
2.4 LIABON ...............................................................................................................................5

SECTION 3  RECOVERY .............................................................................................................6
3.1 ALL CLEAR ..........................................................................................................................6
3.2 DEBRIEF .............................................................................................................................6
3.3 POST-EmerGENCY ACTION ..............................................................................................6
3.4 GEOTECHNICAL REPORT ................................................................................................6

TABLE A4  GEOTECHNICAL EVENT REPORT SHEET ..........................................................9

APPENDIX C .............................................................................................................................10
IMPORTANCE LEVEL ..............................................................................................................10
Section 1 Introduction

1.1 General

The Landslide Action Plan is a sub-plan of the Wollongong City Disaster Plan (DISPLAN). This plan provides for both the Government and non-Government organisations to cooperate in a coordinated manner in the event of an incident or emergency and sets out specific responsibilities and tasks. The structure of this plan is similar to the Wollongong City Local Flood Plan, with particular emphasis on the provision of geotechnical services.

1.2 Purpose

This plan details preparedness, response and recovery operations in the event of incidents or emergencies, resulting from landslide activities in the City of Wollongong.

1.3 Definitions

“Emergency” means an event which endangers or threatens to endanger the safety or health of persons, or damages or threatens to damage property in the Wollongong Local Government Area. Response to the event may be controlled by a Combat Agency or the Wollongong Local Emergency Operations Controller.

1.4 Roles and Responsibilities

This subsection details the roles and responsibilities of the people and/or groups involved in carrying out landslide emergency operations.

1.4.1 Local Emergency Operations Controller (LEOCON)

The Wollongong Local Emergency Operations Controller is primarily responsible for dealing with landslides in the area. The controller will:

a. Maintain an Emergency Operations Centre from which to control landslide investigations and coordinate the actions of supporting agencies.
b. Coordinate a public education program to ensure that the residents in landslide-prone areas are aware of the threat of landslide, particularly following a high-intensity storm event.
c. Activate this landslide action plan.
d. Define and monitor people and/or communities at risk of landslides.
e. Collect, collate and interpret geotechnical intelligence and disseminate it to the appropriate personnel.
f. Direct the conduct of landslide rescue operations.
g. Direct the evacuation of people and/or communities if required.
h. Provide an information service in relation to landslide activity, road conditions and closures, advice on temporary mitigation measures and confirmation of evacuation warnings.
i. Monitor landslide response operations
j. Issue the ‘all clear’ when Emergency Operations have been completed and approval obtained from the Geotechnical Team.

In the event of an emergency, the Local Emergency Management Officer will contact the Council’s Geotechnical Engineer. Other geotechnical personnel, internal or external to the Council, may be called upon to form a Geotechnical Team (GT), depending on the urgency of the incident and on the recommendation of the Geotechnical Engineer.

1.4.2 Geotechnical Team (GT)

The Geotechnical Team, acting in coordination with the LEOCON, is responsible for preparing the following action plan:

(a) Develop a plan for geotechnical and structural stability issues, together with any associated health issues.
(b) Assist Emergency Services by the assessment of reported slope instability.
(c) Visually inspect various sites reported to be subject to instability.
(d) Visually inspect known past problem sites. This involves monitoring and assessment of existing geotechnical instrumentation of SRA, RTA and urban sites.
(e) Determine action plans including referral to Authorities, as appropriate. This involves the preparation of a risk management plan to determine problem sites, in order of priority.
(f) Develop and update a database of the collated information, for documentation and future reference.

1.5 Test and Review Process

This plan will be reviewed by the City of Wollongong Local Emergency Management Committee every two years or following an actual incident/emergency requiring activation of this plan. A meeting will be called, only if necessary, to resolve conflict. All participants should be aware of their roles and responsibilities.

It is the responsibility of participating and supporting organisations to advise the Geotechnical Services Functional Area Coordinator of changes to the Contact Register (Appendix B) as they occur.
Section 2  Response

2.1  Activation

This plan will be activated by Council’s Geotechnical Engineer on advice from the Local Emergency Operations Controller that an event requiring geotechnical services has occurred.

Other appropriate agencies listed in this plan will be advised by the Local Emergency Management Officer as to the location and nature of the threat.

2.2  Communications

During emergencies, to avoid problems with respect to the replication of emergency services response, the following flow sheet (Figure 1) has been proposed. The Geotechnical Services Functional Co-ordinator referred to shall be in direct communication with the Local Emergency Management Officer. All calls to the Geotechnical Branch of the Council requiring geotechnical response shall be referred to the Emergency Operations Centre at level 10 of Council’s Administration Building, 41 Burelli Street, Wollongong.

2.3  Work Methodds

The work method detailed below has initially been drawn up after the 17 August 1998 Wollongong storms.

2.3.1  Immediate Action Plan

1. Obtain detailed rainfall information and forecasts from the relevant authorities.
2. Perform initial site inspections include visits to areas of previously known instability.
3. Assist Emergency Services by assessing reported slope instability.
4. General aerial overview if necessary and available.
5. Collate known and reported sites.
6. Conduct field reconnaissance of sites.
7. Assess risk of slope instability in accordance with nominated guidelines.
8. Monitor existing inclinometers (by UOW, RTA and RSA).
9. Assess appropriate risk management action.
10. Coordinate effort using GT, SRA, RTA, WCC and NPWS.
Figure I  Emergency Services Response Flow Diagram
2.3.2 Risk Assessment

The risk assessment procedure adopted is based on the method developed by Laurie de Ambrosis of GHD-LongMac from the 1997 Thredbo experience. This method is a modification procedure adopted from of Australian Standard AS-NZS 4360 for Risk Management (1995).

The assessment requires a qualitative assessment of the "probability of occurrence of the instability event" (Table A1) in relation to expected conditions and in association with the "consequences of this event" (Table A2) given that the event occurs. In addition, this assessment is split into "person" and "property" components in order to develop appropriate risk management action. The corresponding risk matrix is given on Table A3. It should be noted that under this system various combinations of probability and consequence can have the same risk, e.g. an almost certain but insignificant consequent event (A1) has the same "medium risk" as a rare but catastrophic event (E5).

The provisional risk assessments is made of each site in order to prioritise the action required, with particular reference to "high" risk or greater conditions.

2.3.3 Inspections

The individual site inspections are generally carried out by one of more members of the GT, with appropriate information recorded and photographs taken for documentation purposes. A "Geotechnical Event Report Sheet" has been developed as given on Table A4. It includes landslide information such as Site ID, Location, Description, Jurisdiction, Risk to Person and Property (based on the risk matrix discussed above).

The risk management advice and comments are then conveyed to the EOC meetings during the response phase of the event and/or to other Authorities/Utilities as appropriate.

2.4 Liaison

The Geotechnical Services – Functional Co-ordinator will liaise direct with the LEMO at the Emergency Operations Centre or the headquarters of the controlling combat Agency.
Section 3 Recovery

Procedures as outlined in the City of Wollongong Disaster Plan will apply to the recovery phase.

3.1 All Clear

Following advice from the GT to the LEOCON that the immediate danger to life and property has passed at individual locations, the LEOCON will issue an "All Clear" message signifying that response operations have been completed. The LEOCON will also advise details of arrangements for evacuated residents to return to their homes, if this is possible, or indicate what longer term accommodation arrangements have been made for those unable to do so.

3.2 Debrief

At the time that an "All Clear" message is distributed, the LEOCON will advise all participating organisations of details of response operation debriefing arrangements.

3.3 Post-Emergency Action

Once all reported sites have been individually assessed, prioritised, and compiled in a database, various action needs to be undertaken. These actions can be in the form of:

- Referral to other Authorities and/or divisions/branches of WCC.
- Advice/recommendations to private residents and WCC for further investigation, such as continued monitoring of ground conditions.
- Isolation of the affected area from access in order to protect people from injury.

3.4 Geotechnical Report

A report of the all individual events is to be compiled after the event and handed to the LEOCON and LEMO. The final report shall include:

- The current state of land instability database
- The persons and agencies involved in emergency response
- The resources deployed
- An evaluation of the success of emergency response
- Suggestions for future operations
Appendix A – Risk Assessment

[Note: Tables A1 to A3 have been adopted from Australian Standard AS-NZS 4360 for Risk Management (1995).]

Table A1 Qualitative Measures of Probability of Instability Occurrence

<table>
<thead>
<tr>
<th>Level</th>
<th>Descriptor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Almost Certain</td>
<td>This event is expected to occur in the short-term.</td>
</tr>
<tr>
<td>B</td>
<td>Likely</td>
<td>This event will probably occur in the short to medium term or under adverse conditions.</td>
</tr>
<tr>
<td>C</td>
<td>Moderate</td>
<td>The event may occur within the medium to long-term or under adverse conditions.</td>
</tr>
<tr>
<td>D</td>
<td>Unlikely</td>
<td>This event could occur within the extended long-term or under very adverse conditions.</td>
</tr>
<tr>
<td>E</td>
<td>Rare</td>
<td>This event may occur only in very exceptional adverse conditions.</td>
</tr>
</tbody>
</table>

Table A2 Qualitative Measures of Consequence of Instability Occurrence

<table>
<thead>
<tr>
<th>Level</th>
<th>Descriptor</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Insignificant</td>
<td>Remote likelihood of fatality and/or little damage; low financial loss.</td>
</tr>
<tr>
<td>2</td>
<td>Minor</td>
<td>Unlikely fatality and/or limited damage to part of structure or part of site requiring some stabilisation works; medium financial loss.</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>Possible fatality and/or moderate damage to some of structure, or significant part of site requiring large stabilisation works; high financial loss.</td>
</tr>
<tr>
<td>4</td>
<td>Major</td>
<td>Likely fatality and/or extensive damage to most of structure or extending beyond site boundaries requiring significant stabilisation works; major financial loss.</td>
</tr>
<tr>
<td>5</td>
<td>Catastrophic</td>
<td>Almost certain fatality and/or structure completely destroyed or large-scale instability requiring major engineering works for stabilisation; huge financial loss.</td>
</tr>
</tbody>
</table>
Table A3  Qualitative Risk Analysis Matrix – Level of Risk

<table>
<thead>
<tr>
<th>PROBABILITY of Instability Occurrence</th>
<th>CONSEQUENCE of Instability Occurrence</th>
<th>1 Insignificant</th>
<th>2 Minor</th>
<th>3 Moderate</th>
<th>4 Major</th>
<th>5 Catastrophic</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Almost Certain</td>
<td>Low Risk (L)</td>
<td>Medium Risk (M)</td>
<td>High Risk (H)</td>
<td>Very High Risk (VH)</td>
<td>Very High Risk (VH)</td>
<td></td>
</tr>
<tr>
<td>B Likely</td>
<td>Medium Risk (M)</td>
<td>High Risk (H)</td>
<td>Very High Risk (VH)</td>
<td>Very High Risk (VH)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C Moderate</td>
<td>Very Low Risk (VL)</td>
<td>Medium Risk (M)</td>
<td>High Risk (H)</td>
<td>Very High Risk (VH)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>D Unlikely</td>
<td>Very Low Risk (VL)</td>
<td>Very Low Risk (VL)</td>
<td>Medium Risk (M)</td>
<td>High Risk (H)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>E Rare</td>
<td>Very Low Risk (VL)</td>
<td>Very Low Risk (VL)</td>
<td>Medium Risk (M)</td>
<td>Medium Risk (M)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Legend: Level of risk in AS/NZS 4360:1995:

(H): high risk; detailed research and emergency planning required at senior levels
(S): significant risk; senior management attention needed
(M): moderate risk; management responsibility must be specified
(L): low risk; management by routine procedures.
<table>
<thead>
<tr>
<th>Field</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site ID</td>
<td>[from database]</td>
</tr>
<tr>
<td>Location/Address</td>
<td></td>
</tr>
<tr>
<td>Suburb</td>
<td></td>
</tr>
<tr>
<td>Reported by</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Inspected by</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td></td>
</tr>
<tr>
<td>Category</td>
<td>[1: Historic, 2: Reported, 3: New]</td>
</tr>
<tr>
<td>Event Description (brief):</td>
<td></td>
</tr>
<tr>
<td>Event type</td>
<td>[Landslip, Flood, Seep]</td>
</tr>
<tr>
<td>Risk Assessment</td>
<td>Person [refer matrix]</td>
</tr>
<tr>
<td></td>
<td>Property [refer matrix]</td>
</tr>
<tr>
<td>Responsible Authority</td>
<td></td>
</tr>
<tr>
<td>Action Plan Required</td>
<td>[Y/N]</td>
</tr>
<tr>
<td>Risk Management/Comments</td>
<td></td>
</tr>
<tr>
<td>Attachment Sheet</td>
<td>[Y/N]</td>
</tr>
<tr>
<td>Photos</td>
<td>[Number]</td>
</tr>
<tr>
<td>Action Plan Complete</td>
<td>[Y/N]</td>
</tr>
<tr>
<td>Signed</td>
<td></td>
</tr>
<tr>
<td>Date</td>
<td></td>
</tr>
</tbody>
</table>

**Table A4  Geotechnical Event Report Sheet**
APPENDIX C
EXTRACT FROM BUILDING CODE OF AUSTRALIA – IMPORTANCE LEVELS

EXTRACT FROM BUILDING CODE OF AUSTRALIA
(As Reported in Appendix A of AGS 2007c)

Importance Level – of a building or structure is directly related to the societal requirements for its use, particularly during or following extreme events. The consequences with respect to life safety of the occupants of buildings are indirectly related to the Importance Level, being a result of the societal requirement for the structure rather than the reason per se of the Importance Level.

<table>
<thead>
<tr>
<th>Importance Level of Structure</th>
<th>Explanation</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Buildings or structures generally presenting a low risk to life and property (including other property).</td>
<td>Farm buildings. Isolated minor storage facilities. Minor temporary facilities. Towers in rural situations.</td>
</tr>
<tr>
<td>2</td>
<td>Buildings and structures not covered by Importance Levels 1, 3 or 4.</td>
<td>Low-rise residential construction. Buildings and facilities below the limits set for Importance Level 3.</td>
</tr>
<tr>
<td>3</td>
<td>Buildings or structures that as a whole may contain people in crowds, or contents of high value to the community, or that pose hazards to people in crowds.</td>
<td>Buildings and facilities where more than 300 people can congregate in one area. Buildings and facilities with primary school, secondary school or day-care facilities with capacity greater than 250. Buildings and facilities for colleges or adult education facilities with a capacity greater than 500. Health care facilities with a capacity of 50 or more residents but no having surgery or emergency treatment facilities. Jails and detention facilities. Any occupancy with an occupant load greater than 5,000. Power generating facilities, water treatment and waste water treatment facilities, any other public utilities not included in Importance Level 4. Buildings and facilities not included in Importance Level 4 containing hazardous materials capable of causing hazardous conditions that do not extend beyond property boundaries.</td>
</tr>
<tr>
<td>4</td>
<td>Buildings or structures that are essential to post-disaster recovery, or with significant post-disaster functions, or that contain hazardous materials.</td>
<td>Buildings and facilities designated as essential facilities. Buildings and facilities with special post-disaster functions. Medical emergency or surgery facilities. Emergency service facilities: fire, rescue, police station and emergency vehicle garages. Utilities required as back-up for buildings and facilities of Importance Level 4. Designated emergency shelters. Designated emergency centres and ancillary facilities. Buildings and facilities containing hazardous (toxic or explosive) materials in sufficient quantities capable of causing hazardous conditions that extend beyond property boundaries.</td>
</tr>
</tbody>
</table>

(from BCA Guidelines)
### AGS Suggested Acceptable Qualitative Risk to Property Criteria

<table>
<thead>
<tr>
<th>Importance Level of Structure (1)</th>
<th>Suggested Upper Limit of Acceptable Qualitative Risk to Property (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Existing Slope (3) / Existing Development (4)</strong></td>
</tr>
<tr>
<td>1</td>
<td>Moderate</td>
</tr>
<tr>
<td>2</td>
<td>Low</td>
</tr>
<tr>
<td>3</td>
<td>Low</td>
</tr>
<tr>
<td>4</td>
<td>Very Low</td>
</tr>
<tr>
<td></td>
<td><strong>New Constructed Slope (5) / New Development (6) / Existing Landslide (7)</strong></td>
</tr>
<tr>
<td>1</td>
<td>Moderate</td>
</tr>
<tr>
<td>2</td>
<td>Low</td>
</tr>
<tr>
<td>3</td>
<td>Low</td>
</tr>
<tr>
<td>4</td>
<td>Very Low</td>
</tr>
</tbody>
</table>

**Notes:**

1. Refer to Appendix A, Practice Note (AGS 2007c)
2. Based on Appendix C, Practice Note (AGS 2007c)
3. “Existing Slopes” in this context are slopes that are not part of a recognizable landslide and have demonstrated non-failure performance over at least several seasons or events of extended adverse weather, usually being a period of at least 10 to 20 years.
4. “Existing Development” includes existing structures, and slopes that have been modified by cut and fill, that are not located on or part of a recognizable landslide and have demonstrated non-failure performance over at least several seasons or events of extended adverse weather, usually being a period of at least 10 to 20 years.
5. “New Constructed Slope” includes any change to existing slopes by cut or fill or changes to existing slopes by new stabilisation works (including replacement of existing retaining walls or replacement of existing stabilisation measures, such as rock bolts or catch fences).
6. “New Development” includes any new structure or change to an existing slope or structure. Where changes to an existing structure or slope result in any cut or fill of less than 1.0 m vertical height from the toe to the crest and this change does not increase the risk, then the Existing Slope / Existing Structure criterion may be adopted. Where changes to an existing structure do not increase the building footprint or do not result in an overall change in footing loads, then the Existing Development criterion may be adopted.
7. “Existing Landslides” have been considered likely to require remedial works and hence would become a New Constructed Slope and require the lower risk. Even where remedial works are not required *per se*, it would be reasonable expectation of the public for a known landslide to be assessed to the lower risk category as a matter of “public safety”.

Tolerable risk levels would be one class higher (for example Moderate where Low is acceptable). Consideration should be given by regulators to adopting Tolerable risk to property for Existing Slope and Existing Development situations in a similar vein to the recommended differentiation for risk to life.

After due consideration and taking account of the criteria which were included in AGS (2000,
AGS suggests that for most development in existing urban areas criteria based on Tolerable Risks levels are applicable because of the trade-off between the risks, the benefits of development and the cost of risk mitigation. The recommended Tolerable loss of life risk values for the person most at risk for different situations are shown in Table 1 of the Practice Note.

It is recommended in AGS (2007d) that risks be assessed only for the person most at risk, and not for the average person as suggested in AGS (2000). ANCOLD (2003) reported that the person most at risk always controlled, and that average risks were difficult to define and determine.

The recommended values are higher for existing slopes than for new slopes. This is in keeping with ANCOLD (2003) and general literature on risk tolerability which indicates that persons tolerate risks from existing hazards more than for newly constructed ones. Where development modifies an existing slope, the “new slope” criteria should be applied in accordance with the definitions given for the situation in Table 1 of the Practice Note.

Regulators may decide to apply “acceptable risk” criteria for high consequence cases, such as schools, hospitals and emergency services in recognition of the importance of these structures and as a way of covering societal risk concerns. This is also reflected in the recommended criteria for property loss for different Importance Levels of structures below.

The community may tolerate higher risks from natural hazards than man made hazards (IUGS 1997). Such a consideration by the regulator may result in some natural hazards being tolerated in the face of exceptional expenditure to reduce the risk to tolerable levels. An example of this may be the risks associated with boulder falls from natural cliff lines in a bush reserve adjacent to existing residential development. If the regulator and potentially affected owners were not aware of the circumstances then prior to the landslide risk assessment they would have been “uninformed”. Adoption of such tolerable risks should be made on the basis of an appropriate landslide risk assessment and appraisal of the risk mitigation options.

It is recognized in AGS (2007d) that the recommended criteria are higher than required by NSW Department of Planning (2002). However, those criteria are applied to development such as chemical plants which can be sited in areas where the low risks can be achieved. Urban development is within designated areas, the land owner has no option but to develop at the nominated site (if practical) so the trade-off between risk levels, cost of development and risk mitigation have to be considered. This is a similar situation to dams and is part of the reason ANCOLD have adopted tolerable risk criteria.

Societal Risk may be measured against the ANCOLD (2003) recommended values as given in Figure 4 of Leroi et al. (2005). Reference should be made to ANCOLD (2003) when carrying out such assessments.
EVOLUTION OF SLOPE-LAND HAZARD MITIGATION STRATEGIES AND MEASURES IN TAIWAN

Meei-Ling Lin
Department of Civil Engineering, National Taiwan University, Taipei, Taiwan

Huei-Long Wu
Bureau of Soil and Water Conservation, Council of Agriculture, Taiwan

Abstract: Due to the fragile geological conditions and steep terrain of the mountain area, Taiwan is prone to the debris flow and landslide hazards caused by typhoons, torrential rainfalls, and earthquakes, which often caused severe loss of human lives and properties. In the earlier stage of mitigation strategies of such hazards, emphases were laid on the engineering facilities and land use development. Starting from the past decade, the concepts, such as information system, application of potential analysis and scenario simulation, early warning, land use management, education, insurance, and community involvement, gradually evolved, and have been materialized in the mitigation measures. In recent years, the concepts of ecological engineering and environment protection, sustainable development, and management of watershed were introduced into the formulation of strategies of hazard mitigation. The evolvements of the research development and strategies for debris flow and landslide hazard mitigation measures in Taiwan are introduced in this paper.

INTRODUCTION
Due to the steep terrain and fragile geological conditions in Taiwan, heavy rains carried by typhoon and earthquakes often caused severe slope-land hazard. In 1996, typhoon Herb struck Taiwan, and caused severe debris flow in the Chen-You-Lan river watershed, which killed 27 people and 14 people was missing. The Chi-Chi earthquake with a local magnitude of 7.3 had caused severe ground failures and loss of lives and properties with more than 21,900 landslides involving a total area of more than 8,600 hectares. After the 1999 Chi-Chi earthquake, more than 100 newly occurred debris flow torrents were identified in the central Taiwan, which appeared to have been resulted from landslides caused by the earthquake. In 2001 and 2004 severe typhoons struck Taiwan, both events triggered a great number of slope-land failures and resulted in severe damages. Various mitigation measures were conducted in efforts to reduce the impacts of debris flow and landslide hazard. In 1997 the National Science and Technology Program for Hazard Mitigation (NAPHM) was established, and the concepts of mitigation measures have taken different prospects. Since then, the researches and mitigation strategies started emphasizing measures other than engineering facilities on hazard mitigation. Inventory of slope-land hazard, database, potential analysis, delineation of the affected areas, and assessment of the possible risk caused by such hazards can provide important information for drafting effective and realistic mitigation programs, and to plan ahead for hazard management (Lin 2001). The current status and efforts on mitigation measures and related researches are presented in this paper.

SLOPE-LAND HAZARDS IN TAIWAN
Severe hazards caused by landslides and debris flow have a long record in Taiwan. In 1990, typhoon Ofelia struck Taiwan, and 39 people were killed by the debris flow in the Tung-Man
area, Hualien County. In 1996, typhoon Herb struck central Taiwan, and caused severe hazards especially debris flow in the Chen-You-Lan river watershed, which killed 27 people and 14 people was missing. Some of the major slope-land hazard events are listed in Table 1.

The Chi-Chi earthquake struck central Taiwan on September 21, 1999, with a local magnitude of 7.3, had caused severe ground failures and loss of lives and properties. The reconnaissance called by the National Center for Research on Earthquake Engineering and the National Science Council (2000) documented a total number of 436 items of slope failure. Following the initial reconnaissance, the Council of Agriculture took actions in surveying of landslides based on the aero-photos and SPOT satellite images taken a few days after the earthquake (SWCB 2000). More than 21,900 landslides with a total area of more than 8,600 hectares were identified as shown in Figure 1. The slope failures distributed from Miao-Li to Jai-Yi counties as shown in the figure, and almost all items located to the right or hanging wall of the thrust fault. Among all the cases, two large dip slope failures occurred and both induced dammed-up lakes. In Juo-Feng-Err mountain area, the dip slope sliding involved an area of 200 hectares, and 29 people were buried. In Yun-Lin county, the Tsao-Ling dip slope failure covered an area of 400 hectare with an amount of 120 million m$^3$ of deposit material. Immediately after the Chi-Chi earthquake, more than 100 newly occurred debris flow torrents were identified in the central Taiwan, which appeared to be closely related to the landslides induced by the earthquake. The Xiansane typhoon struck the northern Taiwan in 2000, and triggered severe debris flow hazards which caused more than 10 lives. In 2001, typhoon Toraji induced severe debris flow hazard in Nantou County, and more than 100 people were killed (Chen et al. 2001). Two months later, typhoon Nari struck Taiwan, and caused severe damages to Taipei MRT and about 20 persons died because of debris flow and landslides (Lin et al. 2002b). In 2004, typhoon Mindule along with a weather system dumped more than 2000 mm of rainfall in the central Taiwan. Severe landslides and debris flows occurred especially in the Ta-Jia river watershed; several villages were flooded and damaged. The series of hydraulic power plants along the Ta-Jia river were flooded and damaged (Lin et al. 2004a). One month later typhoon Aery again caused severe damages to northern Taiwan, including a large scale landslide in Tou-Chung, Hsing-Chu County. Several large scale landslides and debris flows occurred in the basin of Shi-Men dam, and the turbidity of water increased significantly. The water supply to the northern Taiwan was greatly affected. In 2004 and 2005, debris flows occurred again in southern and eastern Taiwan; both caused severe damages to properties. In view of the number and scale of the slope-land hazards, it appears to be a continuous issue in Taiwan and effective mitigation measures would be essential.

Table 1: Some case history of severe slope-land hazards before the Chi-Chi earthquake

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Location and type of failure</td>
<td>Pa-gua mountain debris flow</td>
<td>Lin-kou tableland debris flow</td>
<td>Kee-Lung river basin, landslide</td>
<td>Tung-man area debris flow</td>
<td>Nantou County debris flow</td>
</tr>
<tr>
<td>No. of death</td>
<td>1075</td>
<td>17</td>
<td>51</td>
<td>39</td>
<td>41</td>
</tr>
<tr>
<td>No. of injury</td>
<td>295</td>
<td>7</td>
<td>8</td>
<td>10</td>
<td>65</td>
</tr>
<tr>
<td>Affected area</td>
<td>1244 km$^2$</td>
<td>110 ha</td>
<td>4000 ha</td>
<td>&gt;10 ha</td>
<td>600 ha</td>
</tr>
<tr>
<td>No. of houses affected</td>
<td>40428</td>
<td>87</td>
<td>357</td>
<td>35</td>
<td>--</td>
</tr>
<tr>
<td>Property loss (NT$)</td>
<td>&gt;34 billions</td>
<td>10 billions</td>
<td>&gt;10 billions</td>
<td>&gt;10 billions</td>
<td>&gt;100 billions</td>
</tr>
</tbody>
</table>
DEVELOPMENT OF MITIGATION STRATEGIES
In the earlier stage of mitigation strategies of such hazard, emphases were laid on the engineering facilities and land use development. Following the severe hazard caused by the typhoon Herb in 1996, an overall reconnaissance was conducted by the National Science Council (1996). It was found that various mitigation efforts were made by different government agencies with different strategies and techniques, and it was important to integrate these efforts and to implement new concepts and technologies. In 1998, the National Science and Technology Program for Hazard Mitigation (NAPHM) was established, and an office was formed for integration and execution of the program (Yen et al. 1998). The program emphasized more on the non-facility measures, such as establishing potential database, scenario simulation, field monitoring, integration and application of information technologies on formulating policies, and emergency response system. The researches for supporting such measures were promoted and integrated to provide tools for implementation. Further more, starting from the past decade, the concepts, such as early warning, land use management, education, insurance, and community involvement, gradually evolved, and have been materialized in the mitigation measures. In recent years, the concepts of ecological engineering system and environment protection, sustainable development, and management of watershed were introduced into the formulation of strategies on hazard mitigation. Such measures rely heavily on the accurate information derived from database. In order to effectively manage the slope-land hazard, it is important to analyze the potential of hazard, to delineate the potentially affected area, and to simulate the hazard scenario. The current status of the data inventory, potential analysis, scenario simulation, and development in mitigation measures will be introduced in the following sections.

INVENTORY OF SLOPE-LAND HAZARDS AND POTENTIAL MAP
The two major types of slope-land hazards in Taiwan include landslide and debris flow, and more emphases are placed on the debris flow due to the rapid movement and tendency of causing severe damages in the deposition area. The Soil and Water Conservation Bureau (SWCB) started investigation of the potential debris flow torrents since 1991, and 485
potential debris flow torrents were identified and published in 1996. The 1999 Chi-Chi earthquake caused extensive slope failure, which could lead to significant secondary debris flow and slope-land hazard. Immediate action was taken by NSC by conducting field investigations for the central Taiwan to identify the potential debris flow torrents, and models for evaluation of hazard potential and risk were developed (Lin et al. 2000c).

A separate investigation was also conducted by SWCB for Central Taiwan and the number of debris flow torrents increased to 722 based on watershed information. In 2001, typhoon Toraji induced severe debris flow hazard and a comprehensive investigation was conducted, and a total number of 1420 debris flow torrents were identified (SWCB 2002a) as shown in Figure 2. It appeared that follow-up investigation of the potential debris flow torrents was required due to drastic changes in the topography and other related conditions following earthquake and debris flow. In order to clarify the status of the 1420 debris flow torrents, a field investigation was performed for updating and supplementing the database in 2003. To ensure the consistency of the investigation results, a standard format for data collection and operation manual were established, and the field work was conducted accordingly (SWCB 2003a). The example of the database and query system is as illustrated in Figure 3. The inventory and database are currently under constant maintenance and renewal.

The Soil and Water Conservation Bureau (SWCB) started inventory and investigation of landslides since 1990, and a total number of 2,535 landslides with a total area of 8,100 hectares were documented. After the Chi-Chi earthquake in 1999, extensive slope failures occurred. The subsequent ground-based investigation documented 15,977 hectares. In 2001, typhoons Toraji and Nari induced severe landslides and debris flow hazards. The follow-up investigation documented 50,753ha. The total area affected by landslide increased significantly, which appeared to attribute to the impacts of Chi-Chi earthquake.

Figure 2: Distribution of debris flow torrents (Lin et al. 2007)
Since the commencement of the NAPHM program, the Central Geological Survey (CGS) has dedicated significant efforts to inventory and mapping of potential landslide area. The mapping of potential landslide area has been conducted for different region each year from 1999 through 2006 to cover the whole Taiwan, and a total number of 17,940 landslides were investigated covering an area of 52,252ha. The results of each year investigation are as shown in Figure 4, with investigation from year 2002 through 2004 expanded to cover the effects caused by the Chi-Chi earthquake (CGS 2006a). In addition, the investigation of environmental-geological properties of slope-land around the urban areas in Taiwan was initiated under NAPHM from 2002 to 2006. This investigation included the mapping and delineation of potential slope-land hazard area and rock mass rating, the investigated area covered approximately 45% of Taiwan (CGS 2006b). The scheduling and results of the investigation are as illustrated in Figure 5. Currently, the second stage of the investigation has been conducted to cover the high mountain area. All of the data from investigation conducted by the Central Geological Survey were integrated into a web-based GIS system and database as illustrated in Figure 6.
POTENTIAL ANALYSIS AND SCENARIO SIMULATION

For the purpose of slope-land hazard mitigation, it is necessary to evaluate the potential of slope-land failure and delineate the affected areas. The potential analysis of the 485 potential debris flow torrents was first performed by the NAPHM office (Lin et al. 2000b), and subsequently the method for risk assessments of the debris flow torrents was developed (Lin et al. 2000a; 2001b). With the updated database of field conditions of the 1420 debris flow torrents, the potential of debris flow was evaluated and the affected areas was delineated (SWCB 2003a). The potential analysis was performed based on the trend analysis of the updated field data. The related potential geo-environmental factors of debris flow were
selected and assessment model was established. The geo-environmental factors related to debris supply, topography, and water supply condition, which could be readily extracted from the database, were considered. Testing and adjustment of the model were performed on each potential factor to examine the significance of each factor, and the distribution condition versus the different classes within each factor was examined. The rating of classes within each factor and weighting of each factor was assigned accordingly. The debris flow torrents were rated as with high, intermediate, and low potential with frequency distribution breaks at 70% and 30%; the rated results are as shown in Figure 7. A cross comparison of field conditions and verification with some case histories of debris flow hazard with the rated results were made. It was found that the model rating provided consistent and good presentation of the field conditions. The rating for the remedial priority was then made based on the rated potential and the possible damages following the principle shown in Table 2. The delineation of the affected area was performed based on the Soil and Water Conservation Codes (1996) and the influence of site terrain variation and low-lying lands (Lin et al. 2004b). The preliminary map was generated with a fan-shaped area of $105^\circ$ starting from debris overflow point to the downstream direction until reaching the area with slope of $2^\circ$. The potentially affected area was then adjusted based on the field terrain and hazard history. An example of delineation map of the potentially affected zone is illustrated in Figure 8. The field investigation database, torrents distribution, potential rating, and delineation map are integrated into the web-based query system of SWCB.

![Figure 7: The distribution of the final potential rating, with Taipei City as an example](image)

### Table 2: The rating principle of remedial priority

<table>
<thead>
<tr>
<th>Priority rating</th>
<th>Potential rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High</td>
</tr>
<tr>
<td>Damage rating</td>
<td>High</td>
</tr>
<tr>
<td></td>
<td>Intermediate</td>
</tr>
<tr>
<td></td>
<td>Low</td>
</tr>
</tbody>
</table>
In order to understand the hazard caused by debris flow, the simulation of the flow route and impacts of debris are desirable. A procedure using numerical model for simulation of debris flow deposition process was established (Lin 2001) with a commercially available flow model, FLO-2D (O’Brien & Julien 1985). The results of the debris flow simulation provide information of the potential hazardous area and the extent and severity of the debris deposition. With the simulation results, the risk assessment was made to quantify the damages caused by the deposition of debris to the land use, buildings, traffic, and human lives. For rating of damage, different thickness of deposit results in different degree of severity of damage, the thicker the deposit is, the more severe the damage is. An example of such simulation is as shown in Figure 9, and the results of risk assessment could provide information for remedial measures for hazard reduction and preparedness (Lin et al. 2007).

<table>
<thead>
<tr>
<th>Deposition area</th>
<th>4160 m$^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Debris mass</td>
<td>518 m$^3$</td>
</tr>
<tr>
<td>Land use</td>
<td>Residential</td>
</tr>
<tr>
<td>No. of buildings affected</td>
<td></td>
</tr>
<tr>
<td>High</td>
<td>0</td>
</tr>
<tr>
<td>Intermediate</td>
<td>10</td>
</tr>
<tr>
<td>Low</td>
<td>21</td>
</tr>
<tr>
<td>No. of residents affected</td>
<td></td>
</tr>
<tr>
<td>High</td>
<td>0</td>
</tr>
<tr>
<td>Intermediate</td>
<td>40</td>
</tr>
<tr>
<td>Low</td>
<td>84</td>
</tr>
<tr>
<td>Road damaged</td>
<td></td>
</tr>
<tr>
<td>Class</td>
<td>Residential</td>
</tr>
<tr>
<td>Length</td>
<td>190m</td>
</tr>
</tbody>
</table>

Figure 9: Example of scenario simulation and risk assessment for Taipei 040 torrent

The pilot study for development of the landslide potential map has been conducted by the CGS using the event-based statistical method since 2003. The bi-variant analysis was
performed using multiple influence factors, such as slope, aspect, geological formation, and other factors. An example of the event-based potential map of the Kuo-Shin area is as shown in Figure 10 (CGS 2003). Further study is being conducted to generate the potential map covering the whole Taiwan. A pilot study for establishing the methodology of scenario simulation of landslide based on selected cases is also currently underway.

Figure 10: An example of event-based potential map for landslide in Kuo-Shin area (CGS 2003)

FIELD MONITORING OF POTENTIAL SLOPE-LAND HAZARDS
The field monitoring stations of debris flow have been established by SWCB since 1998, and a total of 15 stations were established in debris flow areas with high potential and risk up till 2002. The locations of 13 out of the 15 stations are shown in Figure 11. The instrumentation of the monitoring station include CCD camera for capturing the images of debris flow, rain gauge, geophone for ground vibration induced by debris impact, wire sensor for monitoring flow occurrence, and ultra-sound velocimetry for measurement of flow velocity, and in some locations the pore pressure is measured using piezometer. The data of the monitoring system are acquired by acquisition system in a hut and transmitted through satellite to the information system in SWCB (2002b) as illustrated in Figure 12. The web-based information query has been provided with real-time monitoring of the field conditions as shown in Figure 12. Such system proved to be efficient for emergency response during the typhoon watch period.
The regional monitoring of landslide is being conducted by SWCB using SPOT satellite images, and an automatic process is performed to identify the possible landslide based on pixel variations of images. Ground based investigation is then conducted for verification and management of the slope-land. The field monitoring techniques used on specific landslide site typically include inclinometer, piezometer, extensometer, time domain reflectometer (TDR), and global positioning system (GPS) monitorry technique. The typical layout of field monitoring system is as illustrated in Figure 13. The GPRS technique has been used for data transmission, and web-based query system can be established for real-time monitoring and risk management of landslide especially during the typhoon watch period.

Figure 11: The distribution of field monitoring stations and query system of real-time information (SWCB website)

Figure 12: Illustration of field monitoring system for debris flow
ENGINEERING FACILITIES FOR MITIGATION

Typically the field mitigation facilities of the landslide include two types of works; the first type of work is to reduce the effects of causal factors, such as regrading the slope, reducing the ground water, and reducing slope height, etc., and the second type of mitigation facilities involves using engineering structures to increase the resistance against driving force and reinforcement of the geo-material. Immediately after the Chi-Chi earthquake, it was found that the earthquake caused grounds to loosen and produced fissures and cleavages. New landslides, rock falls, debris flows, and other secondary hazards would be easily triggered by aftershocks or rainfall. Emergency remedial measures and reinforcing engineering works for landslides and debris flow torrents identified with high potential were conducted by the Council of Agriculture before the typhoon season in order to prevent further damages and secondary hazard (Lin et al. 2002a). Such measure was proved effective in reducing the secondary hazard and has been taken since then following significant damages caused by torrential rainfalls or earthquakes. The engineering facilities used for debris flow mitigation involve the principles of reducing potential debris occurrence in the up-stream triggering area, protection against stream banks and bed erosions caused by debris flow in the flow area, and retention or diversion of debris in the down-stream deposition area. Some facilities and their layout are illustrated in Figure 14. The concept of watershed management has gradually evolved and being incorporated into the mitigation strategy. The strategy emphasizes more on the control of debris and the mitigation of the landslides and erosion control in the up-stream triggering area. In addition, the ecological engineering facilities are emphasized and applied wherever appropriate as illustrated in Figure 15. For the infrastructures subjected to landslide or debris flow hazards, the design of the infrastructures has been modified to avoid, reduce, or withstand the impacts of designated scenario caused by the slope-land hazards.

Figure 13: Layout of monitoring system for landslide (modified from Liao & Liao 1996)
EARLY WARNING AND EMERGENCY RESPONSE
The results of risk assessment described in the previous sections could provide detailed information on the scale and extent of damages caused by the scenario. Such information not only is helpful for drafting of an effective and practical hazard mitigation program for slope-land hazard mitigation, but is also applied to support the decision during emergency response. The applications of such information during hazard mitigation processes of hazard reduction, preparedness, emergency response, and recovery are described in the follow.

1. Hazard reduction: Based on the information, the hazard reduction measures taken include setting up of field monitoring and warning system for the area at high risk, sizing and allocation of mitigation facilities, infrastructures built to withstand designated scenario, drafting of insurance policy, delineation of the potential zone, and regulation and management of land use and development.

2. Preparedness: Measures taken during preparedness stage include planning for shelter and routing of emergency evacuation for high risk area, and allocation of resources for emergency response with proper information, and establishing the supporting database
and information system. In addition, people living in the potentially affected zone have been educated and drilled according to the emergency evacuation program. An example for routing of emergency evacuation of the Nan-Shan Village in I-Lan County is shown in Figure 16.

Figure 16: The emergency evacuation map for the Nan-Shan Village, I-Lan County, Taiwan (SWCB website information)

3. Emergency Response: During the response stage, the results of risk assessment could provide reference supporting decision making. The early warning system for potential debris flow area was established based on the rainfall records of case histories (SWCB 2004) as illustrated in Figure 17. The information system supporting decision making for debris flow hazard has incorporated tracing real-time precipitation during typhoon watch period, early warning system, real-time field monitoring, evacuation map and emergency contact (SWCB 2003b). The web-based information system is as illustrated in Figure 18. The emergency evacuation is issued as precipitation record reaching warning criteria during typhoon warning period. The emergency evacuation has been enforced in potentially hazardous areas, which proved to be effective in reducing casualties since typhoon Nari in 2001 (Lin et al. 2002b).

4. Recovery: During the recovery stage, the risk assessment model provides a good tool for efficiency evaluation of the mitigation facilities, and together with planning of evacuation, and land use regulation, the damages caused by the slope-land hazards could be further reduced.
CONCLUSIONS
Due to the fragile geological condition and steep terrain, Taiwan is prone to slope-land hazard triggered by frequent occurrences of typhoons and earthquakes, and an effective mitigation strategy is needed. Since the commencement of the National Science and Technology Program for Hazard Mitigation (NAPHM) in 1998, the mitigation strategy has taken new directions emphasizing more on measures other than engineering facilities on hazard management and reduction. Such measures include inventory of potential slope-land hazard, database, potential analysis, delineation of affected area, scenario simulation, field monitoring, early warning, integration and application of information system to support decision making, and emergency response system. The implementation of such measures has proved to be effective in reducing casualties and impacts of hazard. Furthermore, from the experiences of major hazard, such as Chi-Chi earthquake and typhoon Toraji, it was found that the reconnaissance and follow-up investigations provided important information for...
hazard reduction, and immediate treatment was essential for reducing the impact of secondary hazard. The mitigation strategy has gradually evolved to incorporate concepts of land use management, public education, insurance, and community involvement. In recent years, the concepts of ecological engineering system and environment protection, sustainable development, and management of watershed are introduced. Such measures rely heavily on the accurate information derived from database, and the constant efforts of data updating and maintenance are required. Due to the extensive failures caused by the Chi-Chi earthquake and the natural environment, it is expected that the slope-land hazards could be significant and prolonged, and further mitigation efforts and to plan ahead for hazard management will be essential.

REFERENCES
Proceedings of the 16th Southeast Asian Geotechnical Conference, Commemorative Volume, 403-409.


LANDSLIDE HAZARD ACTIVITIES IN THE UNITED STATES

Peter T. Lyttle
United States Geological Survey

Abstract: Landslides occur in all 50 states of the United States of America and cause on average of 25 to 50 fatalities and damage of at least 3 billion U.S. dollars annually. The indirect socioeconomic impacts are considerably greater, but are not satisfactorily documented. In 2003, the U.S. Geological Survey (USGS), in concert with its many partners in hazard mitigation efforts in the United States, published the National Landslide Hazards Mitigation Strategy - A Framework for Loss Reduction. This document briefly summarized current landslide research, mitigation activities, and defined the roles of various U.S. federal and state agencies, as well as groups within academia and the private sector. It recommended a long-term strategy that called for significantly increased funding, educational outreach, and development of partnerships to strengthen basic research into landslide processes, emergency preparedness, real-time monitoring, and training. The National Research Council (NRC) of the National Academies has subsequently examined this strategy and amplified several of its findings in a report released in 2004. While agreeing with the basic thrusts of the nine major recommendations in the strategy, the NRC asked the USGS and its partners to carefully prioritize activities because of the limited dollars being dedicated to landslide hazard research and mitigation in the U.S. The USGS has also entered into a long-term partnership with the National Weather Service (NWS) to develop a protocol for delivering debris flow warnings in several research areas in southern California. The USGS information about debris flow rainfall thresholds is added to the well established NWS Flash Flood Watches and Warnings system. This has been a success and the USGS/NOAA partnership is exploring several other areas within the United States to expand landslide research and the debris flow warning system. Recently, several state geological surveys, and regional coalitions of counties, have achieved significant milestones in obtaining statewide or regional LiDAR coverages thus enabling these agencies to better inventory, delineate and study deep-seated landslides.

INTRODUCTION
This paper briefly describes the landslide activities of the U.S. Geological Survey (USGS) and its governmental partners at the federal and state levels in the United States. It does not attempt to discuss the excellent landslide research being carried out at many universities and colleges. The USGS leadership role in landslide hazards mitigation arises from the Disaster Relief Act of 1974 (Stafford Act), which delegates to the Director of the USGS the responsibility to issue disaster warnings for an earthquake, volcanic eruption, landslide, or other geological catastrophe.

Annual losses directly attributable to landslides cost the United States a minimum of 3 billion U.S. dollars (USD) and an average of several dozen deaths (Schuster 1996; Schuster & Highland 2001). If indirect costs, such as disruptions to business and transportation, are included, the loss estimates are clearly much larger. In the United States, the earthquake hazard scientific community is able to effectively influence national policy by having their national seismic hazard maps incorporated into national building codes, thus having a major life-saving impact. In the case of landslide hazards, most of the mitigation is accomplished at
the local government level through zoning or permitting regulations. Therefore it is important for the USGS and its partners to look for ways to educate and influence local citizens or community zoning officials. This paper will describe several examples of recent education campaigns.

Due to limited budgets and staff, the USGS and its state partners are able to issue warnings in a very limited number of locations throughout the United States, primarily only in those areas where considerable research has taken place, and where requisite information such as rainfall-intensity duration threshold values, detailed geological maps, and accurate real-time precipitation information is available. This paper will briefly describe progress that the USGS and National Weather Service, an agency within the National Oceanic and Atmospheric Administration (NOAA), have made in implementing a pilot debris flow warning system in southern California.

While landslide inventories of specific areas or modestly-sized regions have been carried out within the United States, there has been no serious attempt to inventory all of the nation’s landslides. This paper will refer to a few of the local examples, and will also describe an exploratory meeting of landslide scientists at the USGS and a number of state geological surveys that was recently held to explore ways to implement common protocols and standards for landslide inventories and databases, so that these worthwhile efforts may be more easily aggregated and used by others.

NATIONAL LANDSLIDE HAZARDS MITIGATION STRATEGY
In 2003, the USGS developed a comprehensive, multi-sector and multi-agency strategy to mitigate landslide hazards for the United States (Spiker & Gori 2003). The strategy focused on nine major areas and suggested that $20 million USD per annum would be needed annual to succeed. The nine areas include:

1. Research
2. Hazard mapping and assessments
3. Real-time monitoring
4. Loss assessment
5. Information collection, interpretation, and dissemination
6. Guidelines and training
7. Public awareness and education
8. Implementation of loss-reduction measures
9. Emergency preparedness, response, and recovery

While there have been some modest increases in federal dollars going toward landslide hazard research at the USGS and a few other federal agencies, significantly less than half of the needed $20 million is being invested today. Thus the Landslide Hazards Program of the USGS and its partners need to carefully prioritize which activities are most critical.

A year after publication of the strategy, the National Research Council (NRC) of the U.S. National Academy of Sciences reviewed the strategy and recommended research priorities for each of the nine areas mentioned above (National Research Council 2004). The most significant recommendation to emerge from this study was to carefully consider and define the roles (both present and future) for each of the Federal, State, local, and private sector partners involved in landslide hazards mitigation to stimulate productive, effective, and
coordinated partnerships. The USGS has taken this advice seriously, and has spent considerable effort strengthening a few critical partnerships during the past two years. To date, these partners include the National Weather Service (NWS) of NOAA, the American Planning Association (APA) and the Association of American State Geologists (AASG). Each of these partners plays a unique role in implementing a successful landslide hazard mitigation strategy. NWS brings the forecasting expertise of Weather Field Offices across the nation, a sophisticated flash flood warning system, which can be adapted to include debris flow and other landslide hazard information and which operates 24 hours a day and 7 days a week. The APA has 36,000 members who make critical planning, zoning, and permitting land-use decisions in their communities. If we are able to get important landslide science information into the hands of these officials when they make their decisions, mitigation is a real possibility. Finally, the AASG represents the state geological surveys of all 50 states and U.S. territories. Even when one combines the landslide hazard workforce of the USGS and AASG, the total number of experts is quite small, less than 50. Thus it is clear that we must leverage our resources, both financial and personnel. AASG has also developed the needed partnerships with local and county emergency managers, and thus is a critical partner in preparedness and response exercises. In 2007, the “Landslide Exchange Group” was formed, which consists of landslide scientists from the USGS, AASG and the Federal Highway Administration. The mission of this group is to develop common protocols for collecting landslide inventory information, and look for ways to better leverage and aggregate our information and make it available on the Internet. The creation of this group was a direct outgrowth of the NRC recommendations (National Research Council 2004).

It has been impossible thus far to implement several key recommendations of the strategy, particularly those that needed large increases in funding. Key among these was a recommendation that the USGS create two competitive grant programs. One would be a Cooperative Landslide Hazard Assessment and Mapping Program to increase the efforts of state and local governments to map and assess hazards in their jurisdictions. The other was a Cooperative Federal Land Management Landslide Hazard Program to increase the capability of the National Park Service, the U.S. Forest Service, Bureau of Land Management, and other such organizations to address landslide hazards through research at USGS. Instead of creating these programs with inadequate budget, the USGS Landslide Hazards Program is being strategic and developing a few modest grants to help individual states and parks. These cooperative projects in a few selected areas of the United States have been quite successful and show the types of excellent research that could be accomplished with the eventual creation of the cooperative grant programs.

**PUBLIC AWARENESS AND EDUCATION**

The USGS is involved in several important education efforts to transfer landslide hazard research and mitigation techniques to the people who most need them. In some cases the USGS works over a period of several years with a specific community to develop tools (e.g., landslide hazard maps or rainfall-intensity-duration thresholds) that will help the community prepare for and mitigate landslide hazards. In other cases, we are partnering with other national and international organizations to provide educational materials to a broad spectrum of the public. In the case of working with a particular community, some lessons are always learned that can be transferred to other communities, but many of the research products are very specific to that area. In the case of more general interest landslide hazards publications, the USGS is trying to reach a much larger audience.
In an attempt to make planning officials more aware of how to use landslide hazard scientific information, the USGS has recently worked with the American Planning Association (APA) to produce a primer on landslide hazards, and to present a number of case studies of how specific communities have successfully incorporated landslide hazard information into their planning and zoning regulations (Schwab et al. 2005). The APA has a membership of 36,000 planners and offers an excellent vehicle to communicate with these influential citizen leaders. This report very successfully “describe(s) where landslide concerns fit into the larger routine of the daily work planners do”. It also recognizes that “planners have a sizeable toolbox for implementing a wide variety of plan policies affecting land-use and development practices”. It tries to answer which tools are the most effective in mitigating hazards in areas with landslide potential, and makes recommendations on how to best ensure compliance with the goals of a community plan. Every state in the United States deals with natural hazard mitigation in its own manner, so this useful book provides examples of how many different communities have responded to the challenges of planning for landslide hazards. The reader, in most cases a county or city planner, can thus pick and choose from a menu of many useful community practices, some of which have been in effect for years. It is critical to learn how lessons learned in one locality or state can be transferred to another, and this educational volume provides insight into that process. Creating this volume with the APA has provided the USGS with access to many planners. However, in order to get the planners to use the information effectively, USGS landslide scientists will need to demonstrate the usefulness of the mitigation tools to several specific communities first.

The USGS is also working under the auspices of the International Consortium on Landslides and cooperatively with the Geological Survey of Canada to create a handbook on best practices for landslide hazard mitigation. This book is being created for the public in general, and will contain straight-forward definitions of landslides, illustrations and photographs to illustrate mitigation methods and tools, and will share some best practices to use around one’s home or business. This book will be published in 2008 and should be relatively easy to translate into a number of languages other than English.

**INVENTORIES AND HAZARD MAPPING**

The USGS has traditionally focused its landslide hazards research in specific geographic areas, such as the Pacific Northwest or southern California, and applied a broad spectrum of our scientific expertise to intensive studies. By taking this approach, and working with state and local partners, USGS is able to make significant advances in landslide process research, inventories of modern and ancient landslides, production of probabilistic hazard maps, and refinement of sophisticated landslide models.

Radbruch-Hall et al. (1983) prepared a landslide overview map of the conterminous U.S. at a scale of 1:7,500,000. This map, which has recently been released in digital format (Godt 1997) depicts areas where large number of landslides exist, and attempts to classify geological units according to high, medium, or low landslide susceptibility. The USGS has compiled larger scale landslide inventory maps for many regions in the U.S. that document locations, types, and in some cases, relative ages of landslides. Some of these inventories document landslides triggered by single events such as a storm or earthquake (Wilson et al. 1985; Ellen & Wieczorek 1988; Jacobson 1993; Morgan et al. 1999; Coe & Godt, 2001; Godt & Coe 2003). These products have proved particularly useful in understanding what geological, topographical or hydrological factors contribute to triggering of landslides, thus allowing better understanding of the landslide process. When these types of inventory maps
have been combined with documented rainfall thresholds, it has been possible to create predictive models for debris flows in specific areas (e.g. Ellen et al. (1997) in the San Francisco Bay region). Other inventories have compiled all landslides in a particular area regardless of age (Nilsen 1971; Brabb & Pampeyan 1972; McGill 1973; Pomeroy & Davies 1975; Colton et al. 1975; Pomeroy 1977). These landslide inventory maps were compiled from aerial photographs with some field checking. Therefore the scale and quality of the aerial photographs determined the size and exact shape of the landslides identified. Some surficial geological maps were created that identify landslides and other kinds of surficial deposits (e.g. Madole 1982). Recently several states and coalitions of counties within states have been able to acquire LiDAR. This has allowed the most modern of inventory maps to show landslide location and extent in exquisite detail, clearly showing head scarps and other detailed features (Wooten et al. 2006). The scientists that have compiled these inventories have never been asked to use a consistent approach or use a protocol established for the entire nation, so the result is that landslide inventories in the United States vary considerably from place to place, and do not even exist in many areas of the country. Recently, the Landslide Exchange Group was formed to explore the possibility of developing common protocols that could be adopted by both the USGS and the state geological surveys, i.e. the two groups responsible for compiling most inventories in the U.S.

Landslide susceptibility maps are another common product produced by the USGS in the last few decades (e.g. Brabb et al. 1972; Pike et al. 2001; Pomeroy 1977). These products provide local governments with more useful information on which to base land-use decisions even though they do not assess the temporal frequency or probability of landslides. In cooperation with the California Geological Survey, the USGS prepared maps showing relative susceptibility of slopes to rainfall-induced debris flows in southern California (Morton et al. 2003). These maps were produced by analyzing six sets of aerial photographs taken during rainy seasons that produced many debris flows, and using digital elevation models of the areas to define the spatial characteristics of the debris flow initiation locations.

WARNING SYSTEMS
Several important efforts to assess the likely frequency of landslides or probabilistic depiction of the likelihood of landslides have been carried out by the USGS (Bernknopf et al. 1988; Mark 1992; Campbell et al. 1998; Jibson et al. 1998; Coe et al. 2000). By tying these sorts of products to real-time precipitation measurements and robust weather forecasts aided by new generations of radar, the USGS and its partner, the National Weather Service, has twice developed debris flow warning systems. The first such attempt began in the San Francisco Bay area during the late 1980’s and early 1990’s (Wilson et al. 1993; Wilson 2004). By carefully documenting rainfall duration and intensity associated with specific landslides, rainfall thresholds were calculated for shallow landslides, mainly debris flows (Cannon & Ellen 1985). These thresholds were further verified in later years with subsequent storms, and refined by documenting antecedent moisture conditions. Similar rainfall thresholds that trigger landslides in other regions of the U.S. have been developed since that time (e.g. Jibson, 1989; Larsen & Simon 1993; and Wieczorek et al. 2000). Even though each of these rainfall thresholds mark a significant advancement, they often can not be extrapolated over larger areas, so much new research in this area is needed in many areas of the U.S.

In 2005, a new memorandum of understanding was established between NOAA and the USGS to establish a Debris Flow Warning System (NOAA-USGS Debris Flow Task Force 2005). The complementary missions of these two agencies were united so that NOAA’s
National Weather Service would supply the USGS with precipitation estimates and forecast grids, and the USGS would operate a debris flow warning system. The concept was straightforward, but the USGS did not have the resources to execute a 24/7 operation. Therefore, a pilot area was chosen in southern California to test the concept and develop an intensive research study area. To further focus the scope of the enterprise, it was decided that recently burned areas would be the selected targets. The USGS has committed to assess the potential for debris flow, to identify infrastructure that may be at risk, and summarize these results in a statement called an Outlook. The USGS also defines, and continually refines, the rainfall intensity-duration warning thresholds. NWS forecasters then analyze measured rainfall and forecast rainfall and issue combined flash-flood and debris flow watches or warnings for the burned areas. Warnings are broadcast through the NWS Advanced Weather Interactive Processing System (AWIPS) to local emergency managers, flood control districts, and the media. At the same time this information is shared with other federal partners through the Common Alerting Protocol (CAP) and Disaster Management Information System (DMIS) recently created by the Department of Homeland Security. To test the success of the system, USGS geologists conduct field reconnaissance checks after each significant storm. If this protocol continues to prove successful, and budgets allow, it may be expanded to other regions of the United States, such as the Pacific Northwest, or the Blue Ridge of the Appalachian mountain system. A much more thorough review of landslide warning systems in the U.S. is given by Baum (2007).

CONCLUSIONS
Much excellent landslide hazard research is being conducted in the United States, by the USGS, its partners in the state geological surveys, academia, and by others. Much of this work is being used to effectively educate community planning officials and the public in general. However, to successfully implement the national landslide hazards mitigation strategy envisioned by the USGS and its partners (Spiker and Gori 2003), significant expansion of our current workforce will be necessary.

REFERENCES


**ACKNOWLEDGEMENTS**

The author wishes to acknowledge the landslide scientists of the USGS, a very small but highly dedicated group of individuals.
CHALLENGES IN LANDSLIDE RISK MANAGEMENT
IN A EUROPEAN PERSPECTIVE

Dinesh Patel
Arup Geotechnics, United Kingdom

Oddvar Kjekstad
Norwegian Geotechnical Institute

Abstract: The European Commission authorized in 2002 a four-year thematic network program under the title GeoTechNet, in which landslide challenges in Europe formed an integral part of a study on natural disasters affecting Europe. The program provided an opportunity to review current practice on the use of landslide inventories, approaches for hazard and risk assessment as well as methods used for mitigation measures. Loss figures in terms of mortality data and economic losses where gathered by the use of questionnaires and public available databases.

Research work done by the GeoTechNet study team estimated that loss of lives caused by landslides in Europe is of the order of 200-250 per year. Economic losses could be of up to 2000-3000 million Euro per year. The study team is of the opinion that number of fatalities and economic losses could be significantly reduced by a more proactive attitude towards landslide risk management than is the present case. The paper contains key issues for improvements suggested by the study team. Those include both actions suggested to be taken by the European Commission and on national level in the different landslide exposed countries.

INTRODUCTION
Landslides are among the most widespread hazards on the earth causing billions of dollars in damage and thousands of deaths and injuries each year around the world. Statistics from The Centre for Research on the Epidemiology of Disasters (CRED) shows that landslide fatalities contributed to about 17% of all other natural hazards in the world (Figure 1).

![Figure 1: Comparison of casualties for different natural hazards (Source: CRED)](image-url)
Historically, Europe has experienced the second highest fatalities and the highest economic losses caused by landslides compared to any other continent. Over the past 100 years, about 16,000 people have lost their lives. With the ongoing climatic changes, continued increase in European population and growing urbanization, it is likely that the consequences of landslide events will be even greater in the years to come.

These facts led the European Commission to initiate a comprehensive investigation in 2002 to quantify the effects of landslides in Europe, identify shortcomings in current risk management practice and propose mitigation measures to reduce future risk. The investigation was undertaken under the umbrella of the thematic EU 5th frame program network GeoTechNet. This paper presents the main findings and recommendations from this investigation which was completed in 2005.

GEOTECHNET

The general objectives of the Geotechnical Thematic Network, GeoTechNet, were:

- to establish and develop a geotechnical European network
- to harmonise geotechnical processes and engineering in key areas across Europe
- to optimise research efforts
- to determine future challenges for Research and Technology Development (RTD) in the field of geotechnical engineering for the benefit of all in the European Community
- to highlight for the European Commission the importance of geotechnics’ role in society when making strategic and policy decisions

Work in the program, which was executed in the four-year period between 2002 and 2005, started with 40 partner organizations from 17 European countries and ended up with 50 participating organizations representing 18 countries at the end. Type of organizations in the program included geotechnical engineers, research organizations, state agencies, universities, consultancy sector, contractors, manufacturers of geotechnical equipment and most importantly end users. The program was led by the Centre for Civil Engineering Research and Codes (CUR) out of the Netherlands and formed a part of the EU program ‘Competitive and Sustainable Growth’.

Activities in the program were carried out in 6 work packages, with separate deliverables. The project website, www.GeoTechNet.com, shows the subject areas dealt with in each of the work packages and the deliverables from some of them. Work package, WP6, dealt with natural disasters and covered the main areas of landslide, flood and earthquake hazards in Europe. The following sections present key findings that were reported on future challenges related to landslide hazards in Europe (Koehorst et al. 2005).

LANDSLIDE INVENTORIES

Landslide inventory forms a crucial part for landslide hazard and risk assessment both on national and sub-national level. It is also important that the inventory provides information to establish a landslide loss frequency diagram for a specific country. Such statistics are useful for comparing risk profiles between different countries and for strategic planning and decision making on national level when risks from different types of hazards need to be compared.

Hence, one of the first tasks carried out in GeoTechNet was to establish the existing status of landslide inventory databases across Europe. Table 1 shows the results of this investigation.
Table 1: Available landslide inventory databases in Europe based on a questionnaire approach in GeoTechNet

<table>
<thead>
<tr>
<th>Country</th>
<th>Material type</th>
<th>Reference</th>
<th>Availability*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Italy</td>
<td>soil and rock avalanches</td>
<td><a href="http://avi.gndci.pg.cnr.it">http://avi.gndci.pg.cnr.it</a></td>
<td>open on request</td>
</tr>
<tr>
<td></td>
<td></td>
<td><a href="http://www.arpa.veneto.it/csvdi/">http://www.arpa.veneto.it/csvdi/</a></td>
<td></td>
</tr>
<tr>
<td>France</td>
<td>soil and rock</td>
<td><a href="http://bdmvt.net">http://bdmvt.net</a></td>
<td>open</td>
</tr>
<tr>
<td>Austria</td>
<td>soil, rock and avalanches</td>
<td><a href="http://www.alpinesicherheit.at">http://www.alpinesicherheit.at</a></td>
<td>open</td>
</tr>
<tr>
<td>Switzerland</td>
<td>soil and rock avalanches</td>
<td>Bundesverwaltung: <a href="mailto:torme@mail.admin.ch">torme@mail.admin.ch</a> SLF, Davos</td>
<td>on request</td>
</tr>
<tr>
<td>Slovenia</td>
<td>soil and rock</td>
<td>On request: Ministry of Environment and Spacial planning</td>
<td>on special request</td>
</tr>
<tr>
<td>Czech Rep.</td>
<td>soil and rock</td>
<td>Czech Geological Survey: <a href="mailto:geofond@sendme.cz">geofond@sendme.cz</a></td>
<td>open</td>
</tr>
<tr>
<td>Sweden</td>
<td>soil and rock</td>
<td>On request: Swedish Geotechnical Institute</td>
<td>no</td>
</tr>
<tr>
<td>Norway</td>
<td>soil and rock</td>
<td><a href="http://www.skrednett.no">http://www.skrednett.no</a></td>
<td>open</td>
</tr>
</tbody>
</table>

Note: Sources of information held in the country provided by the user contacted by GeoTechNet.

This study showed that only a limited number of the European countries actually have an inventory of past landslide events, and where they do exist, they are often not available to all users. Another observation is that the approach used for landslide identification and mapping appears to follow quite different practices across Europe. Of the different inventories available, the one established in Italy seems to be the most comprehensive one. It contains more than 22,000 events, covering landslides in soil and rock in addition to debris flows. The structure of the Italian database may serve as a good practice for what typically should be included in such a database. Figure 2 shows key features from the Italian database, with rainfall and groundwater monitoring statistics (often a major trigger for landslides) forming a crucial link to understanding why landslides occur.

France seems also to be fairly well covered. In the 1990s, Department of Environment in the UK commissioned a comprehensive investigation to establish a database on the distribution of landslides in the country (Figure 3). Unfortunately, this mapping information is not to a sufficient scale as to assist planners and engineers at local scale.

During this study, the importance of maintaining a long term landslide inventory by a responsible organisation adequately funded at national or country level, was found to be severely lacking in many countries.
Geographical distribution of sites historically affected by landslides (green i.e. lighter colour) and flooding (blue i.e. darker colour) – 32,000 landslide events from 1900-2002

Seasonal distribution of landslides (green i.e. lighter colour)

Distribution of Fatalities – red i.e. darker colour (killed); yellow i.e. lighter colour (injured)

Figure 2: Italian model for collecting historical database on landslides (Guzzetti & Tonelli 2004)

Figure 3: Landslide database in Great Britain (Jones & Lee 1994)
LANDSLIDE LOSS FIGURES IN EUROPE

When the European Commission and member countries are to set policy decisions and establish priorities for future investment to mitigate landslide risk, it is vital to understand both loss of life from landslides in Europe, and the direct and economic losses from such events.

Data from the EM-DAT, the OFDA/CRED International Disaster Database in Brussels, shows that Europe has the second highest incidence of landslide causalities of any continent (Figure 4). About 16,000 people have been killed in Europe over the last 100 years. Europe has also the highest incidence of landslide economic losses, averaging about 17 million USD per year (Figure 5).

Figure 4: Number of people killed by landslides from 1903 to 2004 (Source: EM-DAT)

Figure 5: Cost of landslide damage from 1903 to 2004 (Source: EM-DAT)

Among the countries in Europe, Italy clearly experiences the highest number of fatalities (Figure 6). Over the last 100 years, fatalities average about 150 per year (Figure 4) with losses
for Italy alone reported to be 50-60 persons per year. In addition, vast numbers of the population were impacted upon by these events, for each country (Figure 6). Even this number is an underestimate as casualty statistics are often reported in the category of flooding or earthquakes, whilst the direct cause is from a landslide. The actual causality figures referred to above are also likely to be greatly underestimated in the EM-DAT because events with less than 10 persons killed are not reported. GeoTechNet therefore consider the annual casualty figures in Europe are likely to be 200-250 people.

![Graph showing population affected and killed by landslides in various countries](image)

**Figure 6**: Major landslide disasters in Europe (& Russia) during the last 25 years (CRED- 1985 -2006)

Frequently, only the direct losses (e.g. damage to houses/offices) from landslides are reported and very little is known about the indirect losses from landslides. Indirect losses can be widespread, which may include people or businesses not in the landslide area and include interruption to business, transportation networks and communications. It also includes the costs of assistance, storage and accommodation for those displaced by the landslide. These indirect losses can be very large and some examples are given below:

1. Annual landslide costs in Italy, Austria, Switzerland and France are estimated to be 1-5 billion USD (Aleotti & Chowdhury 1999).
2. Munich Re, figures from 2000 show that storm surges, mudflows, and landslides in the Swiss and Italian Alps in one season generated economic losses of about 8.5 billion USD. This was caused by a single event from September 2000 to March 2001 (Geonews 2001).

Large single events can have significant economic impact. For example, the Sarno mudflows in southern Italy led to 25 million Euro in direct costs and an estimated additional 9 million Euro in lost production (indirect losses).

Figure 7 discusses the impact of indirect consequence of landslides on temporary loss of critical lifelines, and the resulting impact on communities from disrupted infrastructure from a single incident in the UK in Autumn 2000.

In the Alpine countries, losses due to snow avalanches induced by winter storm are also significant (Figure 8).
In the extreme Autumn 2000 event in the UK, many incidences of landslide failures induced by heavy rainfall were recorded. This resulted in severe disruption to earthwork infrastructure of railways, including indirect losses due to disruption to trains not running and downtime to business. Ground movements from landslides also affected many properties next to the railway and had to be either repaired or demolished (Source: Birch & Dewar 2003).

Emergency measures had to be implemented by Railtrack to make slopes safe at great cost. Example of this is given at a steep slope in chalk above.

Figure 7: Impact of landslides on critical lifelines from a single incident in Autumn 2000, UK

<table>
<thead>
<tr>
<th>Event</th>
<th>Location</th>
<th>Economic Losses</th>
</tr>
</thead>
<tbody>
<tr>
<td>26 December 1999</td>
<td>Entire country</td>
<td>1,500 million USD</td>
</tr>
<tr>
<td>January - March 1999</td>
<td>15 districts</td>
<td>685 million USD</td>
</tr>
<tr>
<td>July/August 1987</td>
<td>9 districts</td>
<td>800 million USD</td>
</tr>
</tbody>
</table>

Figure 8: Costliest natural disasters in Switzerland (Munich Re MRNatCat Service)
Challenges for Future Policy – Quantifying Loss
The GeoTechNet study showed that decision makers and planners need better and more rigorous estimates of direct and indirect losses from landslide failures. This would then inform judgements on where the highest risk areas are, and the investments needed to mitigate risks or implement emergency recovery procedures at local and national level.

National loss statistics should also consider small landslide events, but this requires cooperation of local authorities, civil protection agencies and municipalities.

Care should be taken that loss figures are properly categorized regarding floods and landslides; in many cases the event is triggered by a flood but the major economic damage is due to the landslide.

TRIGGERING MECHANISMS
Although landslides can be triggered by a number of events, water plays by far the greatest hazard, as shown from the Italian experience. Figure 9 shows the importance of various landslide triggering mechanisms based on the Italian experience. Heavy rainfall is the main trigger for mudflows, the deadliest and most destructive of all landslides.

In Greece, earthquakes are a significant cause of landslides. Papadopoulos and Plessa (1999) have identified 47 Greek earthquakes which caused landslides since 1650 AD. Parise (2000) and Calcaterra et al. (2003) catalogued 13 Italian earthquakes which have triggered landslides since 1125 AD. In coastal areas, liquefaction also played its part in some of these slides.

In summary, the importance of understanding trigger limits of rainfall precipitation, ground water increases during these extreme events and instrumentation and monitoring the ground water table in critical areas, are important factors in understanding the mechanisms of slides, particularly debris flow. Many countries, with possible exception of Italy, are lacking this valuable data and its potential usefulness in mitigating risk, is probably underestimated.

Figure 9: Landslide triggering mechanisms in Italy (Reference AVI database)
LANDSLIDE HAZARD AND RISK ASSESSMENT PRACTICE

Landslide hazard maps do exist in all the exposed countries in Europe, but the approach used, their reliability and availability for the users varies a great deal. Quite a number of the maps are in the form of susceptibility maps rather than hazard maps which identify zones/areas of population and property at risk. Most countries seem to disregard the fact that the correct use of the term hazard is an event with a specified probability of occurrence within a given time and area.

The study team in GeoTechNet observed that use of GIS for susceptibility and hazard mapping is progressing very rapidly in Europe. The advantages with the GIS approach are improved objectivity as well as productivity.

Although long term prediction of landslides based on susceptibility and hazard maps are not specific with respect to time, these maps are crucial for regional planning, transportation and/or other lifeline routings.

When it comes to landslide risk mapping, the practice is not yet so well established and methodologies used varies even more than for hazard mapping. Some countries, like for instance Italy and France, have defined laws requiring that risk maps are being prepared at local level, but the implementation of the regulations was found to be slow. As of 2005, it was found that, in France, only 12% of the risk exposed municipalities had completed the intended work. Reasons for the rather slow implementation of risk mapping in Europe was found to be lack of incentives and lack of practical guidelines and recommended approach to carry out the work.

Methods for risk assessment and risk mapping cover a wide range, from advanced modelling in probabilistic terms to simplified scoring methods. The term "risk" associated with landslide hazards is frequently misused. Risk refers to the hazard and the elements that are vulnerable should the hazard take place. The potential impact (damage) may be in the form of loss of lives or loss of land and property, or both. Risk is strictly the expected degree of loss in a defined area due to a potential damaging phenomenon within a given time period. To comply with this definition, it is also necessary to establish site-specific return periods for the landslide hazard.

Mapping of landslide risk on a national level by following a quantitative or numerical approach is quite demanding. This has however been done successfully by several researchers, and can provide a good illustration of the methodology. An interesting case study was presented for the city of Turrialba in Costa Rica (Van Westen et al. 2002). Another example is part of the Global Disaster Hotspots study where NGI with support from UNEP, Grid Geneva, did a pilot study on the assessment of global landslide risk. In this work, predicted landslide hazard was combined with proximity of vulnerability estimates to obtain risk estimates expressed as risk of loss of life per year per km² (Nadim et al. 2006).

The GeoTechNet study team found that the trend among European countries for solving the risk mapping problem is to apply one or both of the following two approaches:

- Overlay GIS-based landslide hazard maps with elements at risk (population and infrastructure) and obtain potential hotspot "risk areas", where the product of the hazard and the exposure is high. Number of people and extent of elements at risk can then be easily identified and quantified.
- Make use of risk index systems, for instance, in the range from 1 to 5, assessed in a risk
matrix where the two major governing factors within a defined unit area are the level of predicted hazard and the anticipated consequence.

An example of the first mentioned approach is illustrated in Figure 10. It can be used on national level as well as for local investigations.

Example of an Information Management System – Geographic Information System (GIS) for life span management of data and hazard risk is used in private practice on large projects but can be used nationally as well as local level.

GIS Data can be analysed to produce 3D hazard maps draping topography and other data over satellite imagery maps, significantly enhancing hazard and risk mapping.

Figure 10: Use of GIS data and 3D hazard mapping (Source: Arup Geotechnics)
The second mentioned approach, use of risk index or risk classes for zonation of the landslide risk, is a method that is getting more and more attention in many countries in Europe, for instance, Norway and Italy. The method has also been positively received in the developing countries. A typical risk matrix that has been used in Norway is shown in Table 2, where five risk classes are used.

Assessment of the hazard level is done systematically with the use of scores for hazards and weights for the different parameters that are considered to be of most importance. The same applies to consequences. An example of the scoring arrangement for potential quick clay landslide-prone areas in Norway is shown in Table 3 (Lacasse et al. 2004; ICG 2004).

Table 2: Matrix for landslide risk classes adopted in Norway

<table>
<thead>
<tr>
<th>Consequence class</th>
<th>Hazard Low</th>
<th>Hazard Medium</th>
<th>Hazard High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Important</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>Very important</td>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
</tbody>
</table>

Table 3: Example of scoring system for the assessment of landslide consequences (after Lacasse et al. 2004)

<table>
<thead>
<tr>
<th>Elements at Risk</th>
<th>Vulnerability</th>
<th>Weighting VW</th>
<th>Consequence (score)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
</tr>
<tr>
<td>Dwellings, number</td>
<td>4</td>
<td>Dense</td>
<td>2</td>
</tr>
<tr>
<td>Commercial buildings</td>
<td>3</td>
<td>&gt;50</td>
<td>1</td>
</tr>
<tr>
<td>Other buildings</td>
<td>1</td>
<td>High</td>
<td>0</td>
</tr>
<tr>
<td>Road, average no. cars/day</td>
<td>2</td>
<td>&gt;5000</td>
<td>0</td>
</tr>
<tr>
<td>Railway, track priority</td>
<td>2</td>
<td>1-2</td>
<td>2</td>
</tr>
<tr>
<td>Power line</td>
<td>1</td>
<td>Central</td>
<td>3</td>
</tr>
<tr>
<td>Floods/inundation</td>
<td>2</td>
<td>Serious</td>
<td>2</td>
</tr>
<tr>
<td>Sum ( VW* Score)</td>
<td>45</td>
<td>30</td>
<td>15</td>
</tr>
<tr>
<td>% of total score</td>
<td>100</td>
<td>67</td>
<td>33</td>
</tr>
</tbody>
</table>

After the risk classes are established, an activity matrix can be established, showing the type of action needed to be considered for each of the classes. This type of approach is usually most practical to apply on the municipality level. Local participation in the establishment of risk maps has proven to have the best effect and ensures ownership. An example of a risk map developed by this approach in Norway is shown in Figure 11.

Frequently applied methods for assessment of landslide risk on a regional level in Italy has conceptually some similarities with that described method in Norway (Calcaterra et al. 2003).
Figure 11: Landslide risk zonation in a highly landslide prone area, 150 km north of Oslo.

In the Italian legislation, classification of the exposed areas is defined by four classes:

- **R4** = very high risk - possible loss of human lives
- **R3** = high risk - possible serious injuries to people
- **R2** = minor risk - minor damage to buildings and infrastructure
- **R1** = moderate risk - marginal economic and social damage

The risk is defined as the product of the hazard \( P \) and the consequences in terms of damage \( D \):

\[
R(k) = P(n) \times D(m)
\]  

The matrix for calculation of the risk is shown in Table 4.

**Table 4:** The Italian matrix for landslide risk assessment

<table>
<thead>
<tr>
<th>Hazard</th>
<th>( P_3 )</th>
<th>( P_2 )</th>
<th>( P_1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Damage</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( D_4 )</td>
<td>( R_4 )</td>
<td>( R_4 )</td>
<td>( R_3 )</td>
</tr>
<tr>
<td>( D_3 )</td>
<td>( R_4 )</td>
<td>( R_3 )</td>
<td>( R_2 )</td>
</tr>
<tr>
<td>( D_2 )</td>
<td>( R_3 )</td>
<td>( R_2 )</td>
<td>( R_1 )</td>
</tr>
<tr>
<td>( D_1 )</td>
<td>( R_2 )</td>
<td>( R_1 )</td>
<td>( R_1 )</td>
</tr>
</tbody>
</table>
The relative landslide hazard (P) for soil slides-debris/earth flows is defined by three categories:

- P3 = high
- P2 = medium
- P1 = low

Establishment of the relative landslide hazard map is usually carried out following a qualitative approach, where a number of factors expected to control landslides susceptibility are assessed. Among the major factors are geology, geomorphology, slope, land use, soil thickness together with information about historical landslides.

Damage (D) is defined by four levels, D1-D4 (where D4 is very high), is defined as the expected loss of properties or human lives given by the product of exposed value and vulnerability.

Challenges for Europe – Risk Mapping Systems
The use of a risk matrix system in risk mapping of landslides at both national and local level is:
(a) not universally applied across Europe, and certainly not in a standardised manner; and
(b) if it was properly implemented, it would allow the European Commission and member countries to establish better policies and strategic planning to mitigate against future landslide risks

Some aspects of appropriate mitigation measures currently being applied in Europe and strengthening of these measures is described below.

LANDSLIDE HAZARD MITIGATION
Mitigation measures against landslide risk fall into two main camps: active (structural) measures taken to mitigate the risk; or passive (non-structural) measures taken to monitor the risk and take active steps in the event of a problem occurring. These two measures, along with early warning system, are described in more detail below.

Structural Mitigation
This type of mitigation is designed to positively reduce the impact of hazards on people, construction and buildings. Examples include designing engineering structures to withstand landslide damage and the use of measures to hinder and limit the effects of various types of landslides.

In connection with protection of buildings and infrastructure facilities against debris flows, there are a number of methods used, such as:

- Terracing, planting and use of drainage facilities of the mountain slope to protect the slope from moving.
- Restraining of debris flows from damaging roads and by use of gabion or concrete walls at the toe of the slopes.
- Use of deflection dams to channel the debris flow away from installations/settlements (Figure 12).
- Use of check dams for erosion control.
Structural measures, if properly designed, built and maintained, are very effective, although quite frequently rather expensive. Regular monitoring and maintenance are important. The GeoTechNet study team is of the opinion that the current practice of applying engineered structural mitigation measures is sound and well developed in Europe. A number of innovative solutions are applied and they seem to be well fitted to the local conditions. For some of the project examples that the GeoTechNet study team investigated, it could be that a more comprehensive cost benefit analysis would have been justified before deciding upon the final mitigation solution.

![Deflection walls built to protect the settlement at Flateyri, Iceland against avalanches (courtesy of M. W. Lund)](image)

**Figure 12:** Deflection walls built to protect the settlement at Flateyri, Iceland against avalanches (courtesy of M. W. Lund)

**Non-Structural Mitigation**

Land use regulation is widely used as an instrument for reducing the risks, especially in urban areas. Methodologies are readily available for setting the framework for such regulations, for instance, by the use of a three-level risk based zoning mapping approach, which is widely used in many countries in the western part of Europe. Areal planning should be seen in close connection with landslide hazard and risk mapping mentioned previously.

Implementing land regulations (e.g. avoiding development on risky zones) is however very challenging for several reasons, one of them being that there will often be conflicting values about the land use held by different segments of the population. Frequently, pressures of population growth and long standing communities using the land will result in local political or community resistance for land use control. Often the land at risk may be used by poor communities who have decided to live there because they have few alternatives within their economic circumstances. Such settlements are highly vulnerable to landslide disasters and control through “Regulations” cannot solve these problems.

The only instrument that will work is probably full financial support for relocation to less risk exposed areas.

One of the greatest challenges is to set targets and agree upon acceptable landslide risk. A number of countries have in their regulations set guidelines for assessing risk. A typical example for a set of regulations from Norway is shown in Table 5 below.
Table 5: Controlling development in landslide susceptible land

<table>
<thead>
<tr>
<th>Security class</th>
<th>Maximum nominal probability per year</th>
<th>Return period (years)</th>
<th>Type of construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$10^{-2}$</td>
<td>100</td>
<td>Garages, smaller storage rooms of one floor, boat houses</td>
</tr>
<tr>
<td>2</td>
<td>$10^{-3}$</td>
<td>1000</td>
<td>Dwelling houses up to two floors, operational buildings in agriculture</td>
</tr>
<tr>
<td>3</td>
<td>$&lt;10^{-3}$</td>
<td>$&lt;1000$</td>
<td>Hospital, schools, public halls etc.</td>
</tr>
</tbody>
</table>

Linked to the land use regulations are also the quality of and the enforcement of building codes. In the new codes within the European Community there is for instance a statement saying “Areas can only be built on when safety against danger or major inconvenience caused by natural hazards, earthquakes, flooding, landslides, volcanic activities or other kind of natural dangers is acceptable”.

The main challenges associated with non-structural measures are therefore to have proper guidelines and planning documents that considers the risks in detail, as described below:

- Avoid people living in high landslide risk areas (where possible)
- Produce guidance documents for planners when developing on landslide prone sites (e.g. PPG 14 UK guidelines)
- Carry out proper cost-benefit analysis on mitigation measures
- Carry out proper maintenance of protection structures
- Disseminate best practice
- Take steps to ensure that developers are complying with building codes and regulations and that it is being policed at local level

**Early Warning Systems (EWS)**

EWS are monitoring systems designed to prevent events that precede landslides in time to issue an imminent hazard warning. EWS mitigate risk by reducing the consequences. To do this, the system must issue warnings early enough to give sufficient time to implement actions to protect people and their properties.

The five fundamental activities in an EWS are: monitoring/data acquisition, analysis and forecasting, warning and response. Critical aspects of the process are also listed below:

- Effective identification of the risk
- Readiness of the emergency agency
- Well defined evacuation plans by rescue agencies
- Reliable weather forecasting
- Evaluation of trigger thresholds
- Instrumentation and long term monitoring against trigger limits
- Implementation of emergency measures if trigger limits are being approached

The two major concepts used are:

a) **Direct methods** with the use of instruments that directly register debris flow movements or unfavourable displacement patterns in soil or rock masses.
b) **Indirect methods** based on generally observed conditions that based on experience may lead to the triggering of landslides. The methods used range from extremely simple techniques such as educating the public to recognizing and reporting landslide symptoms (e.g. tension cracking, reactivation of spring lines) to more sophisticated meteorological monitoring with the use of rainfall threshold values to give regional warnings.

The GeoTechNet study group found that there are a number of excellent examples on the use of direct methods in Europe. An excellent example is the 1987 Val Pola rockslide in Italy (Figure 13). In this case, a potential rockslide area was monitored by simple displacement instrumentation coupled with an early warning of the community. When the predicted threshold values of displacement were exceeded people in the valley were evacuated saving thousands of lives.

![Figure 13: Val Pola Rockslide – An example of successful early warning system](image1.jpg)

Direct methods are presently used to monitor the threat from a large number of high potential rockslides that are threatening communities, for instance, in Switzerland, Italy, France and Norway. An example from Switzerland is the Ronda rockslide (Figure 14) where the movement pattern of large volumes of rock masses that moves up to 2 cm a year are being closely monitored.

![Figure 14: Ronda rockslide, Switzerland – Application of direct early warning systems](image2.jpg)
At Le Ruines de Sechilienenne near Grenoble in France, a potential rockslide that could involve more than 20 million m$^3$ of mass has been monitored for more than 15 years. In Norway, large communities on the west coast are being threatened by potential rockslides that may contribute to high flood waves in the deep fjords. Another heavily monitored site is the Aaknes rockslide where a comprehensive monitoring and EWS has been recently installed. One example is the heavily monitored break out of 40 million m$^3$ of rock masses that could threaten thousands of people when a flood wave hit the local communities (Blikra et al. 2005). Pioneering work has also been done in Italy to advance technology on measurement techniques by introducing the Synthetic Aperture Radar (SAR). This tool seems to have great potential as documented in several cases in Italy, for instance, in the monitoring of the Ruinon landslide in the Italian Alps (Tarchi et al. 2003).

It is the opinion of the GeoTechNet study group that the use of rainfall threshold values for early warning of landslides (i.e. indirect methods) has not advanced that rapidly in Europe compared with the direct methods mentioned above. The most experienced organizations on this concept is still to be found in Hong Kong and in Rio de Janeiro where the indirect method has been applied for more than 15 years.

However, some promising results from Europe have recently been presented from Italy (Frattini et al. 2004) giving threshold values for triggering debris flows in the Italian Alps. Figure 15, from Frattini et al. (2004), shows how threshold trigger limits may be computed from observation of landslide failures to rainfall data.

It is of the opinion of the GeoTechNet study group that the use of regional/local rainfall threshold values, properly calibrated against landslides deserves wider attention and application. In many cases this approach will call for a better coverage of rainfall monitoring, not only on the flat land areas, but also in the mountain slopes. The ongoing climatic change in Europe may justify further research in this area both on European and member state level.

Figure 15: Example from Valtellina, Italian Alps, 2500 km$^2$ area (Frattini et al. 2004)
(The fitted equation is $I = 12\left(1/D\right) + 0.07$, where $I =$ intensity in mm/hr and $D =$ time in hours.)
Another example is from Norway (Figure 16). This graph shows how critical rainfall is for triggering debris flows and that this may be related to annual precipitation at a particular site (Sandersen 2005).

![Graph showing critical water supply for initiation of debris flow.](image)

Figure 16: Critical water supply for initiation of debris flow from Norwegian experience

CONCLUSIONS AND RECOMMENDATIONS
The GeoTechNet study team concluded that the management of the risk caused by landslides in Europe is a major challenge, which can be significantly improved with proper planning. Exact numbers for fatalities, affected persons, direct and indirect costs have been difficult to assess from available sources. Based on questionnaires and discussions with contact persons in the different countries, the GeoTechNet study team is of the opinion that typical European losses on a yearly basis are of the order 200-250 fatalities and economic loss of the order 2000-3000 million Euro. There is no doubt that the number of fatalities and economic losses can be significantly reduced by a more proactive attitude than is the case today.

As an important stakeholder, it is suggested that the European Commission gives priority to the following actions:

Support the Establishment of European Landslide Hazard Mitigation Program
A great number of the European countries have common problems in dealing with the landslide hazard, but the practice is fragmented across the countries. National policy and guidelines are left to each country to set, often with no single authority responsible for determining the hazard and coordinating the implementation of long-term mitigation strategy.

It is therefore considered that a European National Landslide Hazard Mitigation Program needs to be set up. It could be led by a geotechnical or geological agency (similar to the US Geological Survey). This agency needs to have expertise and experience in landslide research, monitoring, mapping and data collection, analysis, archiving and dissemination. The organization would need to work closely with stakeholders, academia, planners and emergency authorities to ensure that a framework for long-term mitigation strategy for loss reduction is obtained for Europe. Without such a concentrated action in Europe, the GeoTechNet study team is of the opinion Europe will lag behind the rest of the world.
Continue to Support Single R&D Project for the Build up of Expertise on Selected Priority Subjects

Ongoing R&D supported by the European Commission within the landslide related subjects over the last 10 years has been instrumental for the build up of knowledge at national level. To reduce losses, prevention and mitigation are key elements. Landslide mitigation and control will however not be effective unless the triggering and the flow mechanisms for the landslides are properly understood. The study team suggested that further research should be concentrated on these aspects, especially for the rapid moving debris flows that have shown to have such devastating effects. Secondly, it was pointed out that research on rainfall precipitation thresholds that may trigger slides is important. Regionally established threshold values will make it possible to implement early warning systems, specifically for the most destructive shallow debris flows.

On a national level, the GeoTechNet study team identified a number of high priority activities that Governments can initiate and support to improve current practice for risk reduction. Among the most important ones that were highlighted are:

**Institutional Structure**

The governments should secure that appropriate legislation is in place that defines the institutional structure for disaster management and defines roles and responsibilities on national, sub-national and municipality levels.

**Government Incentives**

Government initiatives and incentives should challenge the most risk-exposed communities to establish their Disaster Preparedness Plans, which should include hazard and risk assessment, preparedness and public awareness.

**National Centre**

Strengthening of a national centre for natural hazards has been successful in many countries. These centres should be responsible for long-term strategic planning for dealing with landslide problems at national level.

**Research & Development**

Support national R&D initiatives to improve the national expertise on landslides, preferably coordinated with EC R&D activities.

**Capacity Building (Knowledge Skills)**

Provide financial support for capacity building and training of key personnel on risk management at community level. Also, strengthen educational programs on risk management in schools and at university level.

**REFERENCES**


**ACKNOWLEDGEMENTS**

The authors are grateful for the European Commission for funding the GeoTechNet project in which landslide risk forms a small but important element of the overall project. It thanks all the participants who helped with this project and the contributions made.
LANDSLIDE DISASTER MANAGEMENT IN ITALY

Luciano Picarelli  
Centro Interdipartimentale di Ricerca in Ingegneria Ambientale C.I.R.I.AM.  
Seconda Università di Napoli

Pasquale Versace  
Dipartimento di Difesa del Suolo  
Università della Calabria, Cosenza

Roberto de Riso  
Dipartimento di Ingegneria Geotecnica  
Università di Napoli Federico II

Michele Palmieri  
Settore Programmazione Interventi di Protezione Civile  
Regione Campania, Napoli

Abstract: Italy is one of the most developed countries in the world with the highest risk of landslide. This is due to both a high hazard which features extensive hilly and mountainous areas and the density of population, structures, infrastructures and industries. Historical catastrophic landslides had been testified by old chronicles, but even in the last tens of years, a number of people have lost their lives because of slope movements. Due to the permanent risk posed by landslides and by other natural events such as floods, earthquakes, snow avalanches, etc., some national and regional laws have been approved in the last ten years, giving a strong impetus to the organization and development of the National Department for Civil Protection and other public institutions whose goal is the prediction and the prevention of natural risks. The paper presents a framework of the present situation in Italy concerning the state of the research on the prediction and prevention of landslides and the policy for disaster management.

FOREWORD

Italy is one of the most developed countries in the world with the highest risk of landslide. Historical catastrophic events had been reported in old chronicles and echoes of huge landslides can even be found in literature and poetic masterpieces; unfortunately even today recurrent catastrophes continuously remind politicians and researchers of the problem and the need to make our towns, infrastructures and environment safer. Moreover, new and greater problems are posed by both climatic changes, which are causing an increase in the frequency of some types of landslides, and the unstoppable increase of exposed elements caused by the growth of both the population and the national income. Such problems are exacerbated by the unwillingness of modern society in accepting to be subject to these risks, and also by media campaigns sometimes dictated by unmentionable political reasons. Since also other natural phenomena, such as floods, earthquakes, snow avalanches, volcanic eruptions and summer forest fires cause continuous damages and even casualties, the establishment of organizations devoted to risk prevention and disaster management became necessary. As a matter of fact, after the 1966 Florence overflow, and the 1976 catastrophic earthquakes of Friuli, and Irpinia in 1980, a Ministry for Civil Protection was established in 1982. The first
appointed Minister was Giuseppe Zamberletti who had managed the emergency phase following both earthquakes. Later, a National Department (1992) as well as other regional agencies for Civil Protection have been set up and their goal is the prediction and prevention of natural events as well as the management of emergency. In recent years, the engagement of the Civil Protection has grown rapidly because of the occurrence of other disastrous events, such as the Piedmont flood (1994), the Sarno debris flows (1998), the Versilia flood and landslides (1998), the Stromboli eruption and tsunami (2002) as well as seasonal summer forest fires etc. However, other institutions are engaged in landslide prevention, management and protection of the territory. Concerning the hydro-geological risks, the most important ones are the so called River Basin Authorities, established through a national law promulgated in 1989.

The Italian scientific community is deeply involved in such activities. Referring to landslides, it has been engaged, in the 1980’s and 1990’s, in some nationwide programmes funded by public organizations such as the Ministry for Research and University and the National Research Council. In addition, researchers and scientists provide a significant support to the National Department for Civil Protection and to the River Basin Authorities.

A general overview of the present situation about landslide disaster management in Italy is shortly reviewed in the following. After a short description of the most recent catastrophic landslides in Italy, the first part of the paper illustrates the present situation, with particular reference to flowslides in loose pyroclastic soils and to the mechanical phenomena which govern their occurrence and evolution; the second part is devoted to the strategies which are presently adopted for landslide prevention and risk mitigation.

A SHORT OVERVIEW OF RECENT CATASTROPHIC LANDSLIDES IN ITALY

Landslides represent a permanent hazard in large parts of the Italian territory. Italy is the first country in Europe in terms of the number of fatalities and missing people and the second one in the context of the most industrialised countries in the world. Guzzetti (2000) provides a comprehensive overview of the situation. In the last century, almost 8,000 people were killed by landslides and the homeless or evacuated people were about 100,000 (Figure 1); in the last decade, the average number of casualties has been 26 per year (263 victims). Finally, the cost of repairing damages caused by landslides has been evaluated to be between 1 and 2x10^9€ per year, i.e. 1.5% of the national gross domestic product. After adding indirect costs (loss of productivity, property devaluation, etc.), it could reach 4x10^9€ per year (Canuti et al. 2002). The main causes of this severe situation are:

- the morphology and recent geological history of the peninsula;
- the extent of sedimentary formations constituted by fine-grained soils and sedimentary and igneous-metamorphic highly fractured or weak rocks;
- the high seismicity of most of the country;
- the climatic conditions, which are characterised by relatively long-lasting and/or intense rainfalls, acting as an important triggering factor in the context of the “weak” geomorphological setting of the territory.

The type of landslides is extremely wide because of the variety of geomorphological conditions: rock falls, rock slides and rock avalanches are the main landslides in fractured rocks, while slides, debris flows, flowslides and mudslides are usual movements in granular and in fine-grained soils; also lateral spreads are diffuse. The most frequent catastrophic
landslides are debris flows in pyroclastic soils. In fact, the events of 1910, 1924, 1954 and 1998 also indicated in Figure 1, occurred in the same region, namely Campania (Southern Italy), where these soils extensively outcrop.

The variety of geomorphological situations implies a variety of mechanisms and a wide spectrum of sizes and velocities of the mobilised soil masses. These factors, in turn, affect the measures to be taken for land management and risk mitigation. Large landslides, which can attain tens of millions of cubic metres, are mostly represented by slides, mudslides and lateral spreads in clay, but also debris flows may reach large sizes. The fastest landslides are debris flows, flowslides, rock avalanches, rock slides and rock falls which attain tens of metres per second; slides and mudslides may be quite rapid in their initial stage, reaching tens of metres per minute. On the other hand, active slides, mudslides and lateral spreads can be as slow as a few millimetres per year. Typically, fast mudslides progressively turn into slow long-lasting movements, but the reverse, i.e. a sudden acceleration, can also occur (Picarelli 2001). Of course, the duration and run-out of landslides depends on their velocity: rapid landslides are very short, running hundreds of metres up to kilometres; while slow landslides can last tens of, even hundreds of years, covering only a few tens of metres. As a consequence, the risk is highly variable and the mitigation measures must be carefully chosen as a function of the present or expected landslide behaviour.

Figure 1: Number of fatalities and missing people due to landslides in Italy in the period 1900-2002. The initial value of the cumulated curve refers to the total of fatalities since 1300 (Barla 2005)

Catastrophic landslides are those movements which cause destruction and casualties because of their kinetic energy, which governs the run-out and impact on obstacles. Faster than the speed of a running man, not only do they not leave time for escape, but also no time for setting up any measure to stop the movement.

Examples of Recent Catastrophic Landslides
The geological framework of the peninsula is related to the Alpine-Himalayan orogenesis, during Cretaceous age until Pliocene, the African plate collided with the Eurasian plate. During such a time-span, Alps and Apennines were formed following complex subduction processes and huge overthrusts. As a result of the convergent plate boundary, seismicity and
extensive active volcanism are still widespread.

The history of Italy is constellated by catastrophes caused by landslides. A description of some of the most recent catastrophic phenomena occurred in the main geological settings of Alps, Apennines and Calabrian Arc is shortly reported in the following.

**Landslides in the Alpine Chain**

The Alpine chain can be conventionally subdivided into Western Alps and Eastern Alps. The western part of the chain is mainly made up of plutonic and metamorphic rocks reaching more than 4000 m in elevation. In the Eastern part, dolostones, volcanic and metamorphic rocks prevail, with peaks that seldom exceed 3000 m. The remarkable relief energy, along with intense erosional phases partly related to the last glacial periods, favoured the formation of deep valleys bordered by steep flanks which are subjected to frequent mass movements. Debris flows and rock avalanches, but also huge reactivated relict landslides, pose major problems.

The Vajont slide (1963) and the Val Pola rock avalanche (1987) are among the most destructive events in the recent history. Another catastrophic event, not described here, is the Val di Stava flowslide (Colombo and Colleselli 2004) which involved a tailings dam built on a slope located in the North-Eastern part of the chain to collect the slurry produced by a fluorite mine. The landslide trail covered about 500 m in about 60 seconds, reaching an estimated velocity of about 30 m/s; it impacted against 3 hotels, 52 buildings, 6 industrial buildings and some bridges, killing 269 people.

**The Vajont Rockslide**

On October, 9, 1963, at 6.30 pm, a catastrophic landslide occurred in the Vajont valley (Figure 2a), located in the south-eastern part of the Dolomite area of Eastern Alps, about 100 km north of Venice (Mueller 1964). The mouth of the valley hosts a reservoir with a doubly curved arch dam, which at that time was the world’s highest thin arch dam, reaching about 265 m above the valley floor. During the drawdown of the reservoir, a block of approximately 270 million m$^3$ detached from the right flank of the valley and slid into the lake, attaining an estimated velocity of 30 m/s. As a result of the impact on water, a high wave rose high on the opposite slope, hitting the villages of Casso and Erto; and at the same time, it overtopped the dam crest at heights of 150 to 210 metres, reaching the valley below, where almost 2000 people were killed, mostly in the town of Longarone located about 500 m below while the dam remained unbroken.

Figure 2: (a) The Vajont landslide; (b) the Val Pola rock avalanche
The valley presents a syncline structure made up of Mesozoic sedimentary formations (mainly carbonate, with clay beds). The sliding surface has been located in a thin clay layer interbed. The causes and mechanisms of that event have been debated among the researchers, but a definitive conclusion has not been reached, in particular, some researchers argue that it was the reactivation of an old landslide (Hendron & Patton, 1985), while others suppose that was a first-time movement (Skempton 1966; Petley 1996).

The Val Pola Rock Avalanche
In Val Pola, central Alps, a catastrophic rock avalanche occurred on July, 28, 1987, in an extremely warm period characterised by intense rainfall (Figure 2b). About 40 million of cubic metres were mobilized along the eastern slope of Mount Zandila, as a consequence of the reactivation of a large prehistoric landslide (Crosta et al. 2004). The soil mass rapidly moved downslope into the Adda River valley, forming a landslide dam lake, but part of the debris ran up 300 m rising up on the opposite flank of the valley; in addition, after reaching the bottom of the valley, it moved both upstream and downstream, extending up-valley by 1.0 km and down-valley by 1.5 km. As a consequence, a wave up to 95 m high went upstream for about 2.7 km: 40 people were killed in the villages of Sant'Antonio Morignone and Aquilone located along the Adda river.

Local bedrock consists of isoclinal folds of highly fractured and jointed gneiss intruded by gabbro and diorite and overlain by thin glacial and colluvial deposits (Crosta, 1991). Regarding the triggering factors, Dramis et al. (1995) suggest that permafrost-related processes, such as ground ice melting due to climatic warming and related changes in the groundwater regime, could have played a prominent role. It must be noted that the landslide occurred 8 days after the most intense rainfall period, as typical for very large-scale mass movements that, in that month (July), the cumulated rainfall reached a value three times higher than the monthly mean value; in particular, the return period of the cumulated rainfall between 15 and 19 July has been estimated to be between 200 and 500 years (Crosta et al., 2004).

A description of the activities carried out to prevent a failure of the formed landslide dam is reported later on.

Apennines Chain
The Apennines chain is the backbone of the Italian peninsula, extending, almost continuously, for more than 1000 km from Liguria Region to Sicily. Quite different geomaterials outcrop along the chain. In the northern Apennines, metamorphic rocks and terrigenous formations prevail. In contrast, the central-southern Apennine and the western Sicily are essentially made up of carbonate rocks, which are locally mantled by loose pyroclastic deposits. In the easternmost areas of the chain, turbiditic sequences are present.

All the main settings of the Apennines are affected by a variety of landslides: moderate to slow slides and mudslides in structurally complex formations and very rapid debris flows in pyroclastic soils resulting from the activity of volcanoes concentrated in the cetral-southern part of the peninsula are the phenomena which most severely affect urban settlements and infrastructures. Examples of such phenomena are the mudslides provoked by the Irpinia earthquake (southern Italy, November 1980), the Ancona slide (central Italy, December 1982), the 800 landslides triggered during the Piedmont flood (northern Italy, November 1994), the Covatta mudslide (southern Italy, April 1996), the numerous landslides occurred in Tuscany.
during the Versilia flood (central Italy, June 1998) and the recurrent rainfall-induced debris flows in pyroclastic soils.

The Ancona Slide
Following an intense rainfall period, in the night of December 12, 1982, a huge landslide having a surface area of about 220 hectares developed in the west of Ancona, the capital of Marche Region (Cotecchia et al., 1996). The landslide caused only one fatality, but about 2300 people were evacuated from their homes, about 280 buildings were damaged, 80% of which could not be repaired. In addition, the highway and the railway connecting Bologna to Bari, were either interrupted or badly damaged. The movement has been supposed to be the reactivation of a pre-existing landslide. Historical information concerning movements in the same area since the 18th century are reported by Crescenti (1986).

The geological setting of the area is characterized by an anticline structure, where clay and sand crop out and alternate with marine clay, Plio-Pleistocene in age (Ciancetti et al. 1986). A number of studies based on site investigations have shown the presence of multiple slip surfaces in the Plio-Pleistocene clayey bedrock, the deepest of which exceeds 100 m, which was well below the sea level. A careful analysis of the landslide has been carried out by Sciotti (1997).

The Covatta Mudslide
The Covatta mudslide occurred on April 12, 1996, in Molise, a region characterised by 58 unstable centres over a total of 136 towns, as shown by an official list published in 1904. The landslide is located in the Biferno River catchment, where an inventory carried out in the 1970’s recognized more than 4000 slope movements. The slope had been partially mobilized during 1995 and at the beginning of 1996, when the Biferno river was partially dammed. The April 12 event involved a soil mass bounded by a 500 m long crown. The total length of the landslide was about 1400 m, the accumulation zone is about 200 m wide; its thickness was 12 m in the main track and 17 m in the accumulation zone; the total volume probably exceeded 1 million cubic metres (Figure 3a). The main consequences of the landslide, whose peak velocity attained at least some tens of metres per hour (Picarelli and Napoli, 2003), were the destruction of a viaduct along the State Road No. 647 located along the river and the formation of a landslide dam which created a lake having a volume of about two millions cubic metres (Figure 3b): a vast area was inundated, damaging some private houses.

The landslide involves structurally complex clayey and clayey-marly-arenaceous soils. A comparison among air-photos covering the 1987-1996 time span reveals a complex landslide system made up of active and dormant sub-systems, which were probably reactivated by a roto-translational movement of the bedrock and channelled within the main body. The landslide, and especially its accumulation zone, underwent a further remobilization in May 1997, when the remedial measures built during the emergency phase and a temporary road by-pass were damaged. A short description of the works carried out to avoid further reactivations is reported later on.
Figure 3: (a) Aerial photograph of the Covatta mudslide: the interrupted State Road No. 647 is shown in the uppermost part; (b) the lake formed upstream the accumulation zone.

Landslides Reactivated by the Irpinia Earthquake

On November 23, 1980, a 6.9 M earthquake involved a wide part of Campania and Basilicata Regions, Southern Italy, causing about 3000 casualties, 9000 injuries and severe widespread damage. Among the most relevant consequences of the earthquake, a number of slides and mudslides, mostly reactivated in highly fissured tectonized clay shales, can be mentioned: the largest ones were the Calitri, Buoninventre, Serra dell’Acquara, Torella dei Lombardi, Andretta and San Fele mudslides and the Bisaccia lateral spread (D’Elia et al., 1985).

A large complex slide-earthflow involved the town of Calitri, located on a hill constituted by a Pliocene sequence including grey stiff clays, weak sandstones, sands and conglomerates. Highly fissured tectonized clay shales also outcrop in the urban area. The landslide (approximately 850 m long and up to 100 m deep) caused the death of 7 people and destroyed or badly damaged over 100 houses. Four main elements were recognized within the landslide body (Del Prete & Hutchinson, 1985): a) a major, deep-seated slide, having a volume in the order of 20 million of cubic metres and a depth estimated up to about 100 m; b) secondary retrogressive slides around the rear scarp of the main slide; c) shallow slides in the toe area of the main slide; d) shallow translational mudslides which formed part of the colluvial apron extending down to the Ofanto river.

Two large movements were triggered in the Sele valley (Carrara et al. 1986): the Buoninventre (Figure 4a) and the Serra dell’Acquara mudslide (Figure 4b). The Buoninventre mudslide is a complex movement, with a length of about 3 km and an estimated volume of about 25 million of cubic metres, made by multiple rotational and
translational slides evolving into a mudslide. The landslide involved old landslide debris along with alternating calcarenites, sandstones, marly limestones and clays. The Serra dell'Acquara mudslide occurred in an area where bedrock is made up of Mesozoic carbonate rocks, in tectonic contact with flysch deposits. Following the main shock of the earthquake, a pre-existing mudslide was gradually reactivated over a couple of weeks; finally, a 2500 m long and up to 500 m wide mudslide was mobilized, involving a soil mass of about 28 million of cubic metres. The slip surface was hosted entirely in the flysch sequence, reaching a maximum depth of 33 m.

Figure 4: (a) The crown and the depletion zone of the Buoninventre mudslide; (b) the Serra dell'Acquara mudslide

Another important earthquake-induced movement was the lateral spread which involved the town of Bisaccia, located at the top of a hill consisting of a Quaternary formation, made up of polygenic conglomerate in a sandy-clayey matrix, in turn overlying highly fissured tectonized clay shales (Di Nocera et al. 1995). The lateral spread, already active because of continuous erosion but extremely slow, experienced a moderate acceleration which provoked cracks in the pavements and fissuring in the houses. Further information about the landslide is given below.

Debris flows in the Pyroclastic Soils of Campania

In Campania Region, which the capital is Naples, catastrophic flowslides and debris flows in pyroclastic soils are usual. An incomplete list of some recent events is reported in Table 1, which includes also information about the size of the landslides and the number of casualties.

The basic features of such events are known since long time. In fact, pioneer studies have described similar phenomena occurred in the Sorrento Peninsula, on the Lattari Mts. (Montella, 1841; Ranieri, 1841) and along the Amalfi coast (Bordiga, 1924; Lazzari, 1954; Penta et al., 1954), highlighting the close link existing between rainfall and landslides.

The first series of landslides in the terrible 1997-2006 decade occurred on January 10, 1997, when intense rainfall caused about 400 landslides, some victims and severe damage. Data about those landslides have been published by Calcaterra et al. (1997) and Calcaterra and Santo (2005) which discussed some peculiar aspects of the triggering and evolution phases. On May 5, 1998, and December 16, 1999, a huge number of debris flows were triggered on the slopes of Pizzo d’Alvano (Figure 5a) and Partenio Mts (Figure 5b), causing 165 victims and immense damage to urban centres. Those phenomena have been the topic of several papers which describe the most significant geomorphological, hydrological and geotechnical
aspects of slope failure and post-failure evolution (Versace et al. 1998; Cascini et al. 2000; Olivares et al. 2004). The last killer events occurred on April 5 2005, in Nocera Inferiore, and on April 30, 2006, in the Ischia island.

Table 1: Recent flowslides and debris flows in Campania (Picarelli et al. 2008a)

<table>
<thead>
<tr>
<th>Sector (Figure 13)</th>
<th>Site</th>
<th>Date</th>
<th>Fatalities</th>
<th>Length (m)</th>
<th>Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ba</td>
<td>Ischia</td>
<td>2006</td>
<td>4</td>
<td>450</td>
<td>3*10⁴</td>
</tr>
<tr>
<td>Fb</td>
<td>Cervinara</td>
<td>1999</td>
<td>5</td>
<td>2*10³</td>
<td>4*10⁴</td>
</tr>
<tr>
<td>Fb</td>
<td>Avella</td>
<td>1998</td>
<td>-</td>
<td>15*10²</td>
<td>2*10⁴</td>
</tr>
<tr>
<td>Fb</td>
<td>S. Felice a C.</td>
<td>1998</td>
<td>1</td>
<td>8*10²</td>
<td>3*10⁴</td>
</tr>
<tr>
<td>Fc</td>
<td>Sarno</td>
<td>1998</td>
<td>137</td>
<td>2-4*10³</td>
<td>5*10⁵</td>
</tr>
<tr>
<td>Fc</td>
<td>Bracigliano</td>
<td>1998</td>
<td>5</td>
<td>1-2*10³</td>
<td>15*10⁴</td>
</tr>
<tr>
<td>Fc</td>
<td>Siano</td>
<td>1998</td>
<td>6</td>
<td>14*10²</td>
<td>4*10⁴</td>
</tr>
<tr>
<td>Fc</td>
<td>Quindici</td>
<td>1998</td>
<td>11</td>
<td>1-4*10³</td>
<td>5*10⁵</td>
</tr>
<tr>
<td>Fd</td>
<td>Gragnano</td>
<td>1764-1997</td>
<td>153</td>
<td>2-10*10²</td>
<td>1-6*10⁴</td>
</tr>
<tr>
<td>Fe</td>
<td>Maiori</td>
<td>1954</td>
<td>&gt;300</td>
<td>10³</td>
<td>5*10⁴</td>
</tr>
<tr>
<td>Ff</td>
<td>Massalubreense</td>
<td>1973</td>
<td>10</td>
<td>3*10²</td>
<td>7*10³</td>
</tr>
<tr>
<td>Fg</td>
<td>Avellino</td>
<td>2005</td>
<td>1</td>
<td>4*10²</td>
<td>2*10⁴</td>
</tr>
<tr>
<td>Fh</td>
<td>Montoro Inf.</td>
<td>1997</td>
<td>-</td>
<td>2*10³</td>
<td>3*10⁴</td>
</tr>
<tr>
<td>Fi</td>
<td>Salza</td>
<td>1970</td>
<td>-</td>
<td>4*10²</td>
<td>20*10³</td>
</tr>
</tbody>
</table>

As a result of successive disasters, debris flows became of great scientific interest and several models for the triggering and channelization of flow-like landslides have been developed through geomorphological, hydrogeological and geotechnical approaches. Some data are reported in this paper; further information can be found in papers by Calcatera et al. (2004), Cascini (2005), Olivares & Picarelli (2006) and Versace et al. (2007).

**Calabrian Arc**

Most of the Calabrian Arc is a portion of the Alpine Chain which, following the opening of the Tyrrhenian sea, moved away from the Sardinia-Corse block overthrusting the Apennine chain. The main massifs of the Calabrian Arc are made up of plutonic and highly fractured and deeply weathered metamorphic rocks. This part of the Italian peninsula is one of the most severely affected by earthquakes and landslides.

A crucial role on landslide occurrence is played by weathering (Le Pera et al., 2001), which affects rock masses outcropping in the main mountain ridges. Saprolite and residual soils are subjected to debris flows, while joint-controlled movements are typical of less weathered horizons, where planar or wedge failures may occur. Finally, reactivations of pre-existing, sometimes deep-seated movements, are not uncommon (Calcatera & Parisie, 2005). Some case studies concerning the Sila massif have been reported by Cascini et al. (1994), following pioneer works which had already focused on the critical situation which involves several towns and related lifelines (Almagià 1910).
The Cavallerizzo Landslide

On March 7, 2005, after a period of prolonged rainfall (645 mm in 90 days, about 72% of the mean annual precipitation), the hamlet of Cavallerizzo, in the Cerzeto town, was severely damaged by a complex slide-earthflow shown in Figure 6 (Iovine et al. 2006). Thirty buildings were damaged or destroyed and an important road was disrupted. About 310 inhabitants had to be evacuated to nearby villages.

Several tectonic units, made up of Palaeozoic-Mesozoic metamorphic rocks overlain by clastic terrains (Cenozoic-Neozoic) crop out in the study area. The zone affected by the landslide belongs to a wider large-scale slope movement. The 2005 event has been interpreted as an episode of a long deformation history recorded in the area since the 18th century. Data collected from 1999 and the integrated use of Phrase Structure (PS) and very high resolution optical images (Casagli 2007) revealed movements occurring in the weeks preceding the collapse, with a pre-rupture velocity from 0.8 to 5-6 cm/day.
FACTORS GOVERNING THE VELOCITY OF LANDSLIDES

Varnes (1978) and Hungr (1981) recognized in the velocity of landslide the fundamental factor to consider when dealing with risk assessment and mitigation: its meaning is deemed equivalent to earthquake magnitude. Today, landslide velocity is one of the factors which affects the strategy to adopt for risk mitigation. In fact, while a timely stabilization of slow movements through earthworks, retaining structures, drainage etc., is often possible, in case of rapid landslides, this is very difficult, and the use of passive works or of early warning procedures is very often compulsory (Versace et al. 2007; Picarelli et al. 2007a).

The mechanisms and velocity of landslides depend on several factors related to the morphology of the slope, the structure of the subsoil and constitutive laws of the single layers, and the initial and induced state of stress. Rapid movements are provoked by soil instability caused by the development of an unbalanced force equal to net value between the driving and the resisting force, which is responsible for the acceleration of the landslide body (Leroueil et al. 1996). In the case of long and steep slopes, the velocity attained by the soil mass may be very high, up to tens of metres per second.

The development of an unbalanced force may be the result of either increase of the driving force or decrease of the resisting force. For natural slopes, the first mechanism is not usual, unless slope failure is caused by an earthquake which provokes a cyclic and transient variation of the mean shear stress: a global increase of this is more likely in the case of small and stiff landslides bodies which can be driven by more or less synchronous acceleration in all the points of the soil mass. The second mechanism (decrease of the resisting force) is by far the main cause of instability, which may be provoked by either a drop of the strength parameters or a pore pressure increase (short-term conditions). The first cause, i.e. post-peak reduction of strength parameters, is usual, because natural slopes are typically constituted by brittle over-consolidated (OC) soils or fractured rocks. The case of quasi normally consolidated (NC) soils, as sensitive clays or young pyroclastic soils, is discussed below.

The drop of the shear strength parameters is characterized, first, by vanishing of the cohesion then, by decrease of the friction angle. Vanishing of the cohesion is a consequence of the rupture of interparticle bonds due to cementation or of the decrease of capillary forces (unsaturated soils); in bonded soils, it is sudden because of the stiffness of bonds. Decrease of the friction angle may be due to different causes: in the case of OC clay, it is mainly provoked by rearrangement of soil fabric (dilation) until formation of a slip surface and particle alignment, i.e. transition from a “continuous” to a “discontinuous” medium; in fractured rocks, it is provoked by smoothing of the joint profile during movement. In clay, such a process is quite gradual, because of the magnitude of the shear strain which is required to modify the soil fabric; while along joints it is sudden because of the stiffness of the asperities.

Excess pore water pressure may be triggered because of shear or compression. A build up of excess pore water pressure occurs if the deformation, which leads to slope rupture or which develops after rupture, is rapid enough, thus it is usual in clay, less usual in sand unless rupture is sudden and acceleration is very high as on steep slopes. Liquefaction is a typical mechanism of excess pore water pressure generation due to shear, as in metastable loose sands and silts as well as in quick clays (Castro 1969). It is revealed by continuous pore water pressure increase even after mobilisation of the available strength which implies a decrease in the shear strength along the failure envelope up to an even negligible value. If
the shear strength vanishes, liquefaction turns into fluidization, which is provoked by a complete loss of the strength and transition from a frictional regime to a collisional regime (Iverson 1997). Excess pore water pressure provoked by compressive stress seems to be a typical mechanism in mudslides (Hutchinson & Bandhari 1971; Picarelli 2001). Even though clay does not liquefy, in some cases pore water pressure can attain a value very close to the total stress (Pellegrino et al. 2004). The main causes of build up of excess pore water pressure in mudslides are undrained loading on softened landslide bodies because of accumulation of debris due to erosion or slumping, internal changes of the state of stress provoked by changes in boundary conditions (Picarelli et al. 2005), earthquakes, etc. Excess pore water pressure caused by compression has been supposed to build up also in flowslides (Olivares & Picarelli 2006).

Figure 7 reports the case of infinite slope subjected to shear failure, which applies very well to translational slides, mudslides and flowslides. Since the driving force remains constant during movement, the landslide evolution depends on the mobilized shear strength (resisting force). In fact, depending on soil brittleness (either drained or undrained), the work made by the driving force during movement can be totally transformed into friction at the base of the landslide body (Case A: ductile soil) or into both friction and kinetic energy (Case B: brittle soil). As a consequence, the fastest landslides occur in very brittle soils. It is worth noting that the global energy balance depends also on internal plastic strains, as suggested by the figure, as well as by friction mobilized along internal discontinuities. However, according to some experience on small-scale physical models (Reik & Hesselmann 1977), the part or the energy which is consumed by friction along internal discontinuities should be minor; in principle, the same can be supposed for internal plastic strains, even though for mudslides presenting a thick shear zone, internal plastic shear strains have been supposed to play some role (Picarelli et al. 2008b). Finally, some Authors consider the increase in frictional strength with the displacement rate, due to viscosity, as an additional factor, which is able to contrast any increase in velocity (Corominas et al. 2005). In principle, this phenomenon should essentially characterize fine-grained materials, thus slides and mudslides; unfortunately, the role of the displacement rate on the “drained” shear strength of clay has not been definitely clarified (Kenney 1967; Tika et al. 1996).

In case of brittle undrained failure, the dissipation of positive excess pore pressures during movement can lead to an increase in friction along the slip surface (as for soils presenting a rate-dependent shear strength), and consequent deceleration of the landslide. The magnitude of this phenomenon depends on the rate of consolidation, which is quite slow in case of mudslides. A simplified analysis has been presented by Comegna et al. (2007) for rapid mudslides, whose progressive transition into slow slides has been assumed to be a direct consequence of pore pressure dissipation (Picarelli 2001). Considerations regarding flowslides in essentially granular soils have been reported by Hutchinson (1986).

The model of infinite slope helps very much in the understanding of the mechanics of movement, since it allows to identify the main factors which affect the velocity of landslides. For different morphological conditions, further factors come into play. For instance, if the shear surface is curved (rotational slides), a component of the weight of the soil mass progressively passes from the driving forces to the resisting forces favoring a rapid arrest of the movement (D’Elia et al. 1998).

It is worth noting that, according to different observations, either fast landslides or relatively slow landslides can equally be triggered in apparently similar materials and in similar
geomorphological contexts, because of even little differences in one of the factors discussed above. For instance, in the case of pyroclastic soils, which have been discussed below, even small differences in the density can provoke very different movement patterns and velocity (Picarelli et al. 2008b). The same has been shown by Flechter et al. (2002) who describe two very different landslides mobilized in the same fluvio-lacustrine deposits. Another example reported by D’Elia et al. (1998) concerns the Allori and the Bomba landslides, respectively in highly fissured tectonized OC clay and in slightly fissured stiff clay outcropping in the same open pit mine in Central Italy (Figure 8). The strong difference in their post-failure behaviour can be explained by the different Brittleness Index of the two materials (Bishop 1967), the second one being more brittle than the first one.

As discussed at the beginning of this section, the criteria for prediction of landslides and for risk management strongly depend on their presumed velocity. If failure is supposed to cause a sudden and strong acceleration of the involved soil mass (as in the case of flowslides and of debris flows), monitoring for alerting is not reliable thus more advanced methods are required. The analysis of microseismic waves propagating from fractures in rock masses subjected to pre-failure movements represent an innovative procedure which is being tested in France (Senfaute et al. 2003). This method has been adopted also in the case of fine-grained soils. Some examples are reported by Amitrano et al. (2007). The analysis of the signals captured through optical fibers laid down in the ground is being investigated at the Seconda Università di Napoli. Preliminary data concerning prediction of debris flows in loose pyroclastic soils are encouraging (Picarelli et al. 2007a; Damiano et al. 2008). Alternative procedures to predict rainfall-induced debris flows are based on hydrological methods, i.e. on the relationships between antecedent rainfalls and landslide occurrence. Such methods are well known and have proven reliable in well defined geomorphological contexts (Versace et al. 1998). Recently, the use of more sophisticated approaches which rely on the analysis of slope behaviour are being taken into account (Picarelli et al. 2007a).

\[
F_d = \text{driving force} \\
F_r = \text{resisting force}
\]

Figure 7: Components of energy possessed by a translational slide (Picarelli et al. 2008b)
In contrast with previous case, monitoring possibly associated with rational criteria for decision making is fundamental for slow landslides. Musso & Provenzano (2004) propose the use of neural networks as a reliable and simple way to predict the evolution of slow movements. This method has been successfully applied to the case of a landslide which affects a slope within a reservoir area in Sicily, accounting for the oscillation of the water impounding. An even simpler approach has been proposed by Mandolini and Urciuoli (1997), based on a statistical analysis and interpretation of the recorded values of rainfall height, pore pressure and displacement, and on the extrapolation to the near future of the relationships between these factors.

With these approaches, the behaviour of slopes subjected to slow movements might be predicted with some confidence unless a sudden change of boundary conditions occurs (Picarelli et al. 2004). In fact, there are slow movements which can suddenly accelerate. This is the case of softened fine-grained landslide bodies, which can experience a sudden increase in pore water pressure due to earthquakes or to significant changes in the internal state of stress due to fast loading (Picarelli et al. 2005). As a result, slides can suddenly turn into catastrophic mudslides. The mudslides provoked by the Irpinia earthquake represent good examples. A similar problem is posed by slopes subjected to slow pre-failure movements which can lead to general failure and subsequent acceleration (an example is the Cavallerizzo mudslide). In these cases as in the previous ones, the analysis of historical data can be useful. Some available data are reported in Figure 9 which shows the relationship existing between the current acceleration of the soil mass in the pre-failure stage and the time to failure. The dotted line fits experimental data, while the solid line bounds 95% of them. Using such data, thresholds can be established based on the selection of a time span to slope failure required to guarantee the take off of all the procedures required to assure safety. The use of this or of other similar methods can be supported by automatic readings which are rapidly spreading.

In the intermediate case of impending landslides which might display a moderate to rapid velocity (Cruden & Varnes 1996), monitoring by piezometers, inclinometers and other “usual” instruments is widespread in Italy, even though no rational procedures for alerting exist. Therefore, the decision is highly subjective being generally entrusted to the expertise of the operators.
Figure 9: Relationship between the current landslide acceleration in the pre-failure stage and the time to failure (Pellegrino & Urciuoli 1996)

SOME MECHANICAL ASPECTS OF RAPID RAINFALL-INDUCED LANDSLIDES IN UNSATURATED PYROCLASTIC GRANULAR SOILS

Rainfall-induced liquefied debris flows in unsaturated pyroclastic soils present for Italy a major problem. These phenomena are mostly concentrated in Campania, which is largely mantled by very loose cohesionless volcanic products (Figure 10). As a matter of fact, old chronicles report huge disasters occurred in the past. Because of the extension of the areas covered by pyroclastic soils and the growth of the population and the development of structures and vital infrastructures occurred after the Second World War, recently the situation has become explosive. Today more than 200 towns and tens of thousands of people are judged at risk. As a result, in the last tens of years, hundreds of people have lost their lives.

Pyroclastic soils are the result of the activity of some volcanic centers, the most famous of which are the Phlegraean Fields and the Somma-Vesuvius. The thickness of pyroclastic covers depends on the number and magnitude of the events, on the distance from the event and on the slope angle. On relatively steep slopes, generally it does not exceed a few meters. The stratigraphy, grain size and texture of covers depend on the mechanisms of deposition (flow, surge or fall). Air-fall deposits are layered and consist of alternating layers of volcanic ash and pumice (primary deposits); they are more uniform and seem characterized by a higher porosity than other types of deposits (Picarelli et al. 2006). Interbedded with these layers, paleo-soils and slightly cemented or plastic ashes can be found. Secondary deposits, typically located at the foot of the slopes where they can reach a thickness of tens of meters, consist of reworked material constituted by ash, with inclusions of pumices and, sometimes, of lapideous fragments.

Typically, these materials cover steep slopes. Because of their highly saturated permeability and slope steepness, they present a relatively low degree of saturation, and conversely, a relatively high suction. Therefore, due to the strong influence on slope stability of the
apparent cohesion of soil due to suction, thin granular covers can be stable on slopes having an angle even much higher than the friction angle. Moreover, since deterioration is a negligible phenomenon in these materials, the main cause of rupture is by rainwater infiltration, consequent increase of the water content and decrease of suction. On relatively gentle slopes, full saturation is attained prior to failure.

As shown in Figure 10, soil failure can bring to a rapid landslide if the resisting force cannot balance the driving force. This is generally caused by a drop in the resisting force. In slightly cemented and non-cemented unsaturated granular soil, a post-peak decrease in the shear strength can be provoked by a drop of the cohesion because of rupture of interparticle bonds due to cementation or capillary forces; while in saturated non-cemented soils, it depends on a decrease of the friction. This, in turn, can be a consequence of dilation in dense soil, or of continuing pore water pressure increase in very loose soil and undrained conditions (static liquefaction). A brittle undrained behaviour, which is well known in the case of uniform silty sands (Sladen et al. 1985), has been recognized also for air-fall volcanic ash (Olivares & Picarelli 2001; Damiano 2004; Lampitiello 2004). As a consequence, the peak velocity of landslides occurring on steep and long slopes may attain a high value. As a matter of fact, through the analysis of damages provoked by the impact on structures, Faella and Nigro (2004) argue that the debris flows triggered during the events of 1998, reached peak velocities up to 20 m/s.

Figure 10: (a) Geological map of Campania (Picarelli et al. 2008a); (b) Pyroclastic macro-areas. 1. Pyroclastic air-fall deposits; 2. alluvial deposits; 3. lavas, pyroclastic flows and tuffs; 4. arenaceous conglomerates; 5. marly arenaceous terrigenous deposits with clay interbeds; 6. carbonate rocks; 7. volcanic centres; 8. rivers; 9. pyroclastic air-fall deposits of Phlegraean Fields and Somma-Vesuvius (Picarelli et al. 2008a)
Landslides provoked by liquefaction are called flowslides (Hungr et al., 2001; Hutchinson, 2004); flow-like movements of wet unsaturated soil occurring along very steep slopes are called debris avalanches. The expression, debris flow, is used when the mobilized soil mass enters a gully located on the slope and propagates within this until to the toe of the slope where it spreads forming a fan shaped accumulation zone; in the following, debris flow of liquefied material will be called liquefied debris flows. If the pore water pressure is high enough, the soil can propagate far from the toe, reaching even very large distances. Bearing on theoretical considerations and on the results of some flume tests on volcanic ash, Musso and Olivares (2004) suggest that rapid movements can lead to a complete soil fluidization. Referring to the 1998 events occurred in Sarno and in surrounding areas, this seems to be confirmed by eyewitness and by some movies.

It is worth to remark that liquefaction can occur only if the soil is saturated and presents a void ratio higher than a critical value (Castro 1969). Hence, since pyroclastic sloping grounds are unsaturated, liquefaction is not a phenomenon to be taken for granted (Olivares & Picarelli, 2006). In fact, it can develop only if the amount of infiltrated water is such to completely fill the voids. If this does not occur, i.e. on very steep slopes whose failure occurs before saturation has been accomplished, different types of landslide can be triggered. In fact, small falls typically involve rather tall cliffs and debris avalanches occur on very steep slopes (Picarelli et al. 2008a). In contrast, gentle slopes, i.e. slopes having an angle close to or less than the friction angle, fail only after saturation. If the soil is susceptible to liquefaction, thus it has a void ratio higher than a critical value, and the process of rupture is fast enough to trigger a positive excess water pore pressure, a flowslide can develop; if the soil is not liquefiable, typically a slide takes place. In these cases, liquefaction is a result of failure itself. In other cases, flowslides or liquefied debris flows could be a result of post-failure phenomena provoked by impacts on nearly saturated soils (Cairo & Dente 2003) or by undrained progressive failure (Olivares & Picarelli 2006).

A couple of examples of landslides involving volcanic ashes are reported in Figure 11. Figure 11a shows a debris avalanche which stopped at the toe of the slope: in fact, the houses shown in the lowermost part of the photograph were not touched by the soil mass. In contrast, Figure 11b shows one of the 1998 liquefied debris flows which killed eleven people in the town of Quindici. The soil mass, probably completely fluidized, ran within a deep gully and propagated in the piedmont, rapidly reaching the town.

![Figure 11](image_url)

**Figure 11:** (a) The Monte Spina debris avalanche, Naples (2001); (b) The San Francesco debris flow, Quindici (1998)
LOCATION OF THE AREAS IN CAMPANIA SUBJECT TO FLOWSLIDE HAZARD

According to previous considerations, the areas in Campania subjected to the highest risk are those which are exposed to flowslide, especially when this can turn into a debris flow. A relatively smaller risk is posed by debris avalanches, which occur on very steep slopes but present a shorter run-out.

Since the friction angle of both volcanic ash and pumice typically ranges between 33 and 40°, in the assumption of infinite slope and vertical flow provoked by rain water infiltration, the maximum slope angle compatible with the hypothesis $c'=0$ (i.e. nil suction, or full saturation due to infiltration) is the friction angle of soil. It is worth noting that the adopted assumptions are consistent with typical geomorphological conditions in the parts of the Region which are covered by volcanic soils, which are characterised by long slopes and thin covers overlying fractured rock (generally, limestone). For slopes higher than the friction angle, rupture should occur in unsaturated conditions. However, accounting for the shape of the water retention curve of these soils, for slope angles only slightly higher than the friction angle, the degree of saturation at failure should be very close to 100%. For this reason, following previous considerations, Picarelli et al. (2007b) assume that debris avalanches involve only slopes higher than 45°.

For slope angles less than 45°, rupture should essentially occur for full saturation conditions thus the type of expected landslide is a function of the susceptibility of soil to liquefaction. According to the present knowledge, this essentially depends on grain size, plasticity and initial density. The highest liquefaction potential is possessed by loose uniform silty sand with non-plastic fines. Hunter & Fell (2003) report the grain size curves of soils which proved to be liquefiable, which are quite in a good agreement with those of air-fall deposits in Campania, regardless of the site of deposition (Picarelli et al. 2006). In addition, these materials are non-plastic, with the exception of old weathered deposits, thus the conditions concerning the index properties for liquefaction to occur are fully satisfied. Concerning density, it is well known that liquefaction involves those soils which have a void ratio at rupture well above the Steady-State Line of soil (Castro 1969). For thin covers of air-fall ash, Picarelli et al. (2007b) suggest a void ratio comprised between 1.5 and 1.8 as the one which separates liquefiable soils ($e>1.5-1.8$) from non-liquefiable soils ($e<1.5-1.8$). Available data throughout the region show that air-fall ashes can present values of the void ratio up to 3-4, thus these materials seem to be susceptible to liquefaction, i.e. to generation of flowslides or of liquefied debris flows. In contrast, quite a lower void ratio seems to feature materials deposited by flow and by surge, weathered as well as secondary deposits.

In conclusion, the features of pyroclastic products susceptible to liquefaction should be the following (Picarelli et al. 2008a):

1. a grain size falling in the range of silty sands, which mostly characterises primary air-fall deposits;
2. absence of plasticity, which features all unweathered ash deposits;
3. absence of true cohesion, as in all those deposits which did not experience formation of zeolites (altered deposits);
4. a low density, which is typical of primary air-fall deposits.

The minimum slope angle for which a slope failure can occur (critical slope angle) strongly depends on the permeability of the bedrock. For infinite slope and vertical seepage, which can take place only for highly pervious bedrock, the critical slope angle is equal to the
friction angle of soil. For impervious bedrock, the minimum value of the critical slope angle may be obtained assuming the groundwater level at the ground surface and seepage parallel to slope. For $\gamma_{sat}=15$ kN/m$^3$, the slope is definitely stable only for angles less than $13^\circ$-$15^\circ$. Therefore, if soil is susceptible to liquefaction, a flowslide is theoretically possible for $\beta$ roughly comprised between $15^\circ$ and $45^\circ$ depending on the nature of the bedrock.

All these considerations match experience. Figure 12 reports the slope angle of the sources of flowslides in Campania in the last years. A major factor is the nature of bedrock, which can be constituted by either fractured limestone (pervious bedrock) or flysch (impervious bedrock). For pervious bedrock, about 80% of the liquefied landslides occurred for angles in the range $30^\circ$-$45^\circ$ and 90% in the range $30^\circ$-$50^\circ$, while for impervious bedrock, about 80% of the landslides occurred for angles in the range $15^\circ$-$35^\circ$ and 95% in the range $10^\circ$-$35^\circ$.

Figure 12: Slope angle in source areas of flowslides in pyroclastic soils (Picarelli et al. 2008a)

Using the available data, a qualitative macro-zoning of potential sources of flowslides and liquefied debris flows can be carried out (Figure 13). The same approach can be employed for more detailed zoning, through the results of investigations providing a detailed stratigraphy of the site and laboratory tests aimed at quantifying the liquefaction potential of soil (Picarelli et al. 2007b). An example is reported in Figure 14a, which concerns an area close to Sarno, where some debris flows were triggered in May 1998. The location of the landslides and of a number of cracks recognized on the ground surface after the event are reported in Figure 14b. It is shown that the potential sources of liquefaction correspond to those areas where debris flows were really triggered or cracks appeared on the ground surface.
Figure 13: Areas exposed to the highest risk of catastrophic rainfall-induced landslides (Picarelli et al. 2008a). 1. Potential sources of flowslides in pyroclastic materials. 2. Sector. 3. Flowslide or group of flowslides. 4. Boundary of municipality. 5. Pyroclastic air-fall deposits of Somma-Vesuvius and of Phlegraean Fields

POLICY, LAWS AND REGULATIONS
Previous sections clearly show the complexity of the landslide hazard scenarios in Italy. In recent years, such situations has been exacerbated by the strong increase of exposure in the most of the territory which has been the cause of continuous occurrence of catastrophic events, and by a growing awareness of the population. This stimulated an increasing engagement of institutions in the research of systems for land protection and risk mitigation. In fact, over the last years considerable changes have been made in the policy of protection against natural hazards. This enabled to obtain significant results which are described in the following part of the paper.
Figure 14: (a) Slopes judged to be susceptible to flowslide in an area close to Sarno; (b) location of cracks and rainfall-induced debris flows in May 1998

Up to the 1980’s, the only existing general regulations were those promulgated at the beginning of the century. The first important law, in 1908, concerned land protection in hilly and mountainous areas. In particular, it included a list of the towns to be abandoned or subjected to stabilization works because of landslides. Such a list has been progressively integrated with further towns. Another law promulgated in 1923 imposed hydro-geological planning control on land use.

However, the societal awareness of natural risks has gradually changed. Initially, the population was quite fatalist, considering natural risk as something unavoidable to be accepted. This was a consequence of the relatively high return period of catastrophic events in the same area, which leads to forget what happened in the past, especially when nothing occurred passing from a generation to another. Recently, this attitude has been changing because of a number of reasons such as the growing number of catastrophic events caused by the increase in vulnerability of exposed areas, the role played by the mass media which offer ample account of catastrophes occurring in other parts of the world which had never been covered before, the increasing sensibility to safety, which is related to the growth in individual and social wealth. As a result, in previous years the production of laws and regulations is greatly increased. As usual, the main adjustments always follow natural disasters.

In 1952, a law was promulgated concerning the measures to adopt in dealing with the consequences of a disastrous overflow of the Po river (1951). In 1966, as a consequence of the flooding of Florence, a special scientific Commission coordinated by Prof. Giulio De Marchi was appointed. The Commission collected an extraordinary number of data, giving a precise account of the hydro-geological state of the Country. In addition, it prepared a “General plan and guidelines for the hydraulic regulation and control of rivers, including the measures required”. The main propositions of such a plan was the establishment of “Basin Authorities”, each one located in a specific hydrographic system, with the task of predisposing the Basin Master Plans. These plans had to include in-depth analysis of hydro-geological risks and proposals for mitigation, based on identification of risk prone
areas, risk assessment, adjustment of mitigation strategies and mitigation procedures. The plan had also to cover measures to deal with water resources and protection. Such a noticeable work convinced the public opinion and the Parliament to provide, after an extremely long and complex debate, a “National Law for Soil Protection” which was promulgated about twenty years after (National Law No. 183, 1989). The main reason of such delay was the political situation of the Country, which at the time was abandoning its State organization in favour of a local politics of ample regional autonomy. Such a law can be considered a very innovative one; in fact, it put the Country in the van to the point that only in 2000 the European Parliament emanated a directive imposing all its Members to adopt a norm analogous to the one contained in Law No. 183.

The main goal of the Law No. 183 was to safeguard human lives, properties and the environment from floods and landslides, according the Basin Authorities the role of coordinating centres of the State and of the Regional Institutions for the hydro-geological risks. That law was followed by regulations and guidelines promulgated at different times, especially on the occasion of catastrophic events, as for the Laws of 1998, after the Sarno event, and of 2000, after the Soverato overflow. In particular, the first one imposed that the Basin Authorities map the areas at risk and adopt measures for risk mitigation. In addition, it stimulated urgent programs for the mitigation of hydro-geological risk.

The Civil Protection began effectively carrying out its functions only after the 1976 and 1980 earthquakes. However, only after a long time, the Law No. 225, 1992, promoting the establishment of the Civil Protection National Service, was approved. The basic elements of such law were:

- The setting up in Rome of the National Department for Civil Protection.
- The attribution to Civil Protection of the forecast, prevention and rescue deeds.
- The classification of catastrophic events in three groups according to their magnitude and the establishment of three corresponding levels of intervention: 1) at a local level; 2) through coordinating institutions; 3) with extraordinary powers and means.
- The delegation to the Council of Ministers of the power to command the state of emergency that allows to operate notwithstanding the existing provisions of the law: it provides for the coordination of the National Service and the promotion of civil protection services through the National Department for Civil Protection.
- The division of the roles to be carried out by all involved institutional subjects.
- The establishment of a National Group for Protection against Hydro-geological Disasters (G.N.D.C.I.) and a “High Risk Committee”. The G.N.D.C.I., which was already operative since 1986, has been an important link between the civil protection system and the scientific community.

A number of consecutive norms has divided the national system of Civil Protection into separate levels: national (through the National Department for Civil Protection), regional, municipal and inter-municipal. In case of disastrous events, the National Department for Civil Protection has to assess in a very short time the magnitude of the event and its possible consequences and the capability, or not, of the local structures (at the municipal level) to face it. In case of National emergencies, the National Department for Civil Protection takes on all the responsibilities. This strategy became more and more efficient, giving rise to a system which at present appears to be working satisfactorily.

In principle, the activities to be carried out for risk mitigation are the followings:
The forecast which consists of identifying risk areas. It is carried out through by the Basin Authorities which predispose maps of the areas subject to landslide risk.

The prevention which is mostly aimed at offering real-time forecasting of the events through a system of national warnings and immediate realization of safeguarding measures for the population and evacuation of risk areas. The national warning system is coordinated by the National Department for Civil Protection. Functional Centres operate basically in each Region, managing automatic rain gauges and hydrometer networks and, very soon, a system of meteorological radars. Functional Centres launch warnings according to forecasting or measurement of occurring rainfall with regards to the areas subject to occurring or foreseen events. Warnings are sent off to the involved Municipalities which activate plans of emergency as shown below.

The rescue starts after the event. It either involves local structures or other territorial institutions according to the event. In the case of severe events and high numbers of casualties, the Council of Ministers proclaims the state of emergency. A Delegate Commissary is appointed, who is usually the Governor of the hit region or the Mayor, as in the case of large Municipalities. The Commissary must guarantee aids to the population and indemnities for suffered damages. He also oversees and manages the overcoming phase of the emergency. In fact, he must predispose in a short time plans of intervention for recovery and rehabilitation of the damaged areas and, once received the necessary funding, carry them through.

The normative frame in Italy is, however, much more complex than what has been briefly drawn here, considering that the normative sources are at least three. There is the European level where Italy figures along with 26 more countries: the European Parliament emanates directives to which all the member Countries have to accommodate their national legislations. There is the national level with the laws emanated by the National Parliament and the directives emanated by the Government. There are also the Regions which have to add the laws and national directives to their regulations and promulgate autonomous laws concerning their areas of competence which includes the soil protection. The Civil Protection system includes also the whole of the Municipalities which must uniform themselves to national and regional norms. Such a complex and contradictory framework has nonetheless allowed, in the past twenty years, for significant improvement in the Civil Protection and soil protection.

STRATEGIES FOR RISK MITIGATION

The strategy adopted in Italy to tackle heavy hydro-geological events is quite complex and articulated. Its fundamental elements are research, organization and planning. From the 1970’s onward, research has been increasingly enhanced through strategic projects funded by the National Research Council. Such an impulse increased even more after 1986. Thanks is due to the establishment of the G.N.D.C.I. which promoted a research line on “Prediction and Prevention of Highly Risk Landslides”, which involved about thousand of researchers providing beneficial effects even at the operative level. This activity was stimulated even more by the establishment of the Basin Authorities and of the National Department for Civil Protection. In addition, the Law No. 183 produced a significant change in the activities concerning land planning.

Today risk analysis is carried out through quite a uniform approach all over the territory, regardless of the Basin Authority involved. The risk is defined through the classical
expression proposed by Varnes & IAEG (1984): \( R = E \cdot V \cdot H \). Since the quantitative assessment of the single factors present in such an expression (elements at risk, \( E \), vulnerability, \( V \), and hazard, \( H \)) is quite complex, approximate methods are used based on a qualitative evaluation of some of the factors concerned (as hazard) and the consequent establishment of risk classes. Starting from the simplified procedure originally proposed by Versace et al. (1995), risk assessment is essentially carried out as in the following:

1. hazard assessment and classification in a range comprised between \( H_0 \) (negligible hazard) and \( H_3 \) (very high hazard);
2. assessment of the magnitude of the expected landslide based on its presumed velocity; the magnitude is classified between \( I_0 \) (negligible) and \( I_3 \) (for velocity higher than \( 10^{-4} \) m/s);
3. assessment of the exposition to risk; the factor \( E \) ranges between \( E_0 \) (desert or unproductive areas) and \( E_3 \) (urban areas, industrial and commercial settlements and so on);
4. assessment of the potential damage which ranges between \( D_0 \) (negligible) and \( D_3 \) (risk for human lives, potential damage to buildings and infrastructures, potential interruption of economic activities). The potential damage is obtained from the hazard, \( H \), and the magnitude \( I \), through the following matrix:

<table>
<thead>
<tr>
<th></th>
<th>( I_0 )</th>
<th>( I_1 )</th>
<th>( I_2 )</th>
<th>( I_3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( E_0 )</td>
<td>( D_0 )</td>
<td>( D_0 )</td>
<td>( D_0 )</td>
<td>( D_0 )</td>
</tr>
<tr>
<td>( E_1 )</td>
<td>( D_0 )</td>
<td>( D_1 )</td>
<td>( D_1 )</td>
<td>( D_2 )</td>
</tr>
<tr>
<td>( E_2 )</td>
<td>( D_0 )</td>
<td>( D_1 )</td>
<td>( D_2 )</td>
<td>( D_3 )</td>
</tr>
<tr>
<td>( E_3 )</td>
<td>( D_0 )</td>
<td>( D_2 )</td>
<td>( D_3 )</td>
<td>( D_3 )</td>
</tr>
</tbody>
</table>

5. risk assessment: the risk is classified in a range comprised between \( R_0 \) (negligible) and \( R_3 \) (unacceptable). It is obtained from the potential damage, \( D \), and hazard, \( H \), using the following matrix:

<table>
<thead>
<tr>
<th>( H )</th>
<th>( D_0 )</th>
<th>( D_1 )</th>
<th>( D_2 )</th>
<th>( D_3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( H_0 )</td>
<td>( R_0 )</td>
<td>( R_0 )</td>
<td>( R_0 )</td>
<td>( R_0 )</td>
</tr>
<tr>
<td>( H_1 )</td>
<td>( R_0 )</td>
<td>( R_1 )</td>
<td>( R_1 )</td>
<td>( R_2 )</td>
</tr>
<tr>
<td>( H_2 )</td>
<td>( R_0 )</td>
<td>( R_1 )</td>
<td>( R_2 )</td>
<td>( R_3 )</td>
</tr>
<tr>
<td>( H_3 )</td>
<td>( R_0 )</td>
<td>( R_2 )</td>
<td>( R_3 )</td>
<td>( R_3 )</td>
</tr>
</tbody>
</table>

Such a methodology with further simplifications has been adopted by the Law and is used by the Basin Authorities. The Law identifies four risk classes between \( R_1 \) and \( R_4 \), where \( R_4 \) being the maximum risk level. The entire territory has been consequently classified, and land planning must now take into account of the risk concerned.

Risk mitigation is carried out by so called “non-structural” and “structural” measures. Non-structural measures are those which do not require any type of work for risk mitigation, but impose restrictions in land use and emergency plans. The restrictions in land use concern the areas which are classified in the categories \( R_3 \) and \( R_4 \), where new constructions or even the enlargement of existing buildings is forbidden. The emergency plans are discussed in next
section. Structural measures are all those works which are purposely built for slope stabilisation or for protection of exposed goods. Earthworks, drainage, retaining works, anchors and nailing are currently used for slope stabilisation; often, a combination of different measures is employed. Passive works, such as barriers, check dams, sedimentation basins etc. are adopted in those areas where active measures are too expensive or cannot be used because of complex morphological conditions or of other restrictions.

Some examples of structural and non-structural measures adopted to tackle catastrophic landslides are reported in the following. The measures adopted in the Sarno area after the events of May 1998 are described by Versace et al. (2007).

The financing for the measures for risk mitigation is sometimes irregular. In fact, different and often uncoordinated financing sources exist. Some funds are directly disbursed by the Ministry for the Environment, some by Basin Authorities, others are the result of agreements between the Government and the Regions. It has to be remarked that an important percentage of public financing concerns post emergency measures which take place in the follow-up of catastrophic events. In such cases a special structure is set up at the hands of a Delegate Commissary who manages the reconstruction and safeguarding of the hit zone and nearby areas with major investments, especially if compared to ordinary ones.

A complex and long debate has been taking place in Italy about the role and the actual impact of such emergency measures. A major point concerns the fact that an efficient coordination between ordinary and emergency activities has not yet been attained. Undoubtedly however, an articulated and complex system of efficient measures as those adopted after the Sarno event could have never taken place without the extraordinary management of all available resources as in that emergency phase (Versace et al. 2007). Surely, that event has constituted a turning point for Civil Protection in Italy.

**EMERGENCY PLANS**

The Emergency Plan describes the actions to undertake before and after catastrophic events. In Italy, these plans may concern the national, regional and municipal level. The key points of the plans which are adopted at a municipal level are shortly described in the following.

Generally, three phases are accounted for before the occurrence of the event: the “advice”, the “watch” and the “warning” phases; an “emergency” phase is activated after the event. The Plan adopts a series of actions to be carried out in each phase. The main components of the plan (Figure 15) are:

- A *monitoring network* to capture in real time relevant hydrological and, when available, geotechnical parameters, to be sent to the Regional Functional Centre. In the majority of cases, the monitoring network includes only automatic rain gauges.
- A *data-bank* containing mathematical models is able to compare the monitored values of the selected factors to the threshold values. In the easiest case, the “advice”, “watch” or “warning” levels are associated to pluviometric thresholds corresponding to an established rainfall duration (Versace et al. 1998). In more complex situations, numerical models can be adopted, which is able to simulate the phenomena occur at the slope up to collapse. According to the collected data and elaboration carried out through such models, the Functional Centre sends off a warning to all involved municipalities.
- *Risk scenarios* define the areas which can be subjected to landslides and the typology of
the expected event, especially concerning the velocity of the movement.

- A network of observers empowered to immediately reach the areas exposed to risk and assess, based on experience, the seriousness of the expected event. The observers contribute to a much more precise definition of risk scenarios based on documentation filed by Basin Authorities. As an example, a so-called Field Survey Team has been established in the Sarno area where a number of geologists and engineers with ample expertise and knowledge of the territory can reach through safe paths and a few designated points of observation to transmit information in real time. This is extremely important in case of warning. Such positive experience has progressively spread all over the country.

![Figure 15: Main components of the emergency plans](image)

Prominent information to decide the activation of the Plan can be obtained according to the observations carried by the Field Survey Team and the warning launched by the Functional Centre. Decisions are to be made by the Major. As a general rule, the advice and watch phases are automatically activated. The “warning” phase is more delicate and requires careful assessment through information provided by the Field Survey Team.

The Plan concerns the activities to be undertaken in the different phases of the event and the assignment to people of concerned tasks. During the “advice” phase it is necessary to activate a municipal operative office and guarantee a continuous presence of people, and an efficient connection by phone or fax to all civil protection structures. It is also demanded that the people assigned to the management of the subsequent phases be put on alert. The “watch” phase activates direct observations of the phenomenon, carried out by the Field Survey Team or by other personnel appointed to the task. It also requires an increase of personnel which would be responsible for different tasks concerning road conditions, communication, health, volunteering etc. Personnel responsible for successive operation phases are also to be alerted. During the “warning” phase the number of competent personnel is enlarged while decisions are taken concerning possible direct actions in the critical areas, carrying out, if necessary, immediate operations for risk mitigation or realizing safeguarding operations such as closure of risk areas, partial or total evacuation of vulnerable areas etc. Such activities are the duty of Fire Brigade and police corps with the aid of volunteers. The “emergency” phase takes over after the event and is aimed at securing the population, taking care of the wounded and
collecting the victims in the case of such a tragic occurrence. To realize timely and effective rescue, it is demanded that the Plan be extremely detailed with regards to the management and operation teams and means (excavators, scrapers, trucks and so on) as well as the population of the exposed areas, with special attention to those who, due to age or health condition, are not autonomous and cannot take refuge on their own.

At a more general level, the Plan must appoint a number of personnel to each task (direct observation and field survey, participation in the promoted activities etc.), it must also identify in advance a number of safe areas for collecting evacuated people and ensuring their shelter in tent camps, as well as the rescuers accommodation and resources storage. It is worth noting that the Plan can assume very specific aspects according to the size of the town, the number of people living in the exposed areas, the presence of other types of risk (floods, sea-storms etc.), but it must always be consistent to the reference frame drawn above.

The municipal Plan of emergency is aimed at managing emergencies which the Municipality can face with the aid of nearby municipalities and the Province to which it belongs. When the event is spread and severe, direct intervention of the National Civil Protection is demanded.

**SHORT EXAMPLES OF DISASTER MANAGEMENT**

As shown, the measures to adopt for risk mitigation and disaster prevention strongly depend on the features of the existing or expected landslide. In particular, it is of prominent importance to account for the expected velocity of the movement. In the following, some cases of landslides characterised by different displacement rates will be shortly reviewed: the extremely slow Bisaccia lateral spread, the extremely slow to slow Campagna slide, the moderate to rapid Covatta mudslide and the extremely rapid Val Pola rock avalanche and Sarno debris flows.

In the case of extremely slow movements (slides, mudslides, lateral spreads), especially if they involve large soil masses, living with landslide is a natural style of life for several communities.

A typical example is the one of Bisaccia town, already mentioned above (Di Nocera et al. 1995). Movements are governed by continuing erosion in the basal clayey formation around the hill on the top of which Bisaccia stands. This phenomenon is responsible for a deficient pore water pressure regime due to the relatively “fast” total stress decrease with respect to the dissipation rate of induced negative excess pore water pressure. The interpretation of data suggests that the present estimated velocity (a few millimetres per decade) will not significantly change in the near future, unless erosion will strongly accelerate, but even in this last case, the deformation process should be delayed by building up of higher negative excess pore water pressure. Periodic changes in boundary conditions are provoked by seismic activity which in that area is significant. As a matter of fact, investigations carried out after the Irpinia earthquake (1980) showed that the interaction of the hill with underlying stiff clays provoked a temporary increase of pore water pressure in the clay deposit located below the hill, which gave rise to a post-seismic subsidence monitored during 8 years after the quake (Fenelli et al., 1992). The peak settlement rate due to subsidence was something more than one centimetre per year, thus not so fast to convince the population to evacuate. It is to mention that according to the 1908 Law, in 1930, Bisaccia was included among the towns to be abandoned, but also in that occasion a part of people refused the evacuation and remained home.
The Bisaccia case is representative of a typical problem in Italy, concerning the stability conditions of old urban centres built on slopes, which has been the topic of several national conferences and workshops (Firenze, 1980; Bologna, 1986; Spoleto, 1993) and of special investigations and researches. The cases of Agrigento, Pienza, Orvieto, San Leo and of other famous old towns are well know, but there are numerous other towns which present similar problems. In the case of the magnificent Orvieto, central Italy, which rises on a tuff slab resting on overconsolidated clay, important stabilization works have been carried out on both the slab and the basal clay deposit. In several other cases, the town has been abandoned because of large landslides which have involved a great part or the entire urban centre, as for the town of Craco, abandoned in 1963, and of the wonderful Civita di Bagnoregio. Some aspects concerning the mechanics of slow landslides and the interaction of these with settlements and man made works have been discussed by Picarelli et al. (2004).

A very different case is described by Picarelli & Simonelli (1991) who describe the Campagna slide consisting of calcareous debris which moves over a clay deposit with a practically constant displacement rate of about 1 cm/yr, recorded over the period 1988-1989 (Figure 16). The landslide body is extremely pervious and can rapidly drain downslope the infiltrating rainfall water; hence, ponding water cannot form and the pore water pressure measured at the base of the landslide is very small, being subjected to only minor changes. As a consequence, the movement is primarily driven by creep along the slip surface and will presumably continue in the future with the same velocity. As a matter of fact, structures built on the slope ten years before the installation of the instruments present only very small fissures. Only a monastery built about some hundreds of years before, at the time of monitoring appeared badly damaged because of large cracks. In fact, it was subsequently demolished.

These examples demonstrate that living with landslides is sometimes possible, especially when hydro-geological and mechanical conditions are favourable and the size of the landslide body is such that structural measures are too expensive. Moreover, the cost to definitely arrest extremely slow landslides can be very high; in addition, a complete stop is not always sure because of the role of both residual internal strains and creep along the slip surface, which can display an only slightly smaller rate than the velocity monitored before stabilization (Russo et al. 2004).

In the case of slow to moderate landslides, active measures as earthworks, drainage and retaining woks are often employed. Some of these measures can be adopted also for stabilization of landslides attaining velocities up to metres per day (from moderate to rapid landslides). This is a special situation, because the displacement rate (tens of metres per hour) is not such to provoke victims but the movement can provoke strong damages.
in order to avoid a further complete damming of the Biferno river and favour a progressive
decrease of the average velocity. This goal has been progressively achieved through two
separated design phases: in the first one (Picarelli & Napoli, 2003), a row of structural wells
was built in the accumulation zone with the aim to avoid the river damming, while the run-off
converging in the landslide basin was intercepted by ditches and drainage trenches located
upslope the main crown; finally, the continuous alimentation of the right side of the landslide
body caused by a smaller mudslide was definitely stopped by some retaining works. In the
second phase, the crown of the landslide, which represents the major alimentation zone, has
been reshaped, a large part of the softened clay which constitutes the depletion zone has been
substituted with a better material (gravel or compacted clay), while deep drainage works have
been carried out in order to maintain the pore water pressure at low values; finally, a retaining
structure located at the mouth of the depletion zone has been built in order to cut the flow of
clay downslope, avoiding any increase of the state of stress and consequent excess pore
pressure generation in the material which fills the track (Figure 17). Today, the landslide is
moving very slowly.

In the case of very rapid to extremely rapid landslides (velocity from metres per minute to
metres per second), active works cannot be carried out during the event. Since for many
reasons, a definite evacuation of people at risk is not easy, when possible, passive works are
carried out in order to protect the life of people, structures and infrastructures. However,
today, the idea to set up early warning systems is being seriously accounted for, mostly where
the construction of passive works is impossible for technical or economic reasons (Picarelli et al. 2007a).

A good example of risk management concerning a huge very rapid landslide is the Val Pola landslide briefly illustrated above. It was one of the first cases in which Civil Protection had the opportunity to deal with a catastrophic landslide testing the efficiency of the system.

The meteorological event which triggered the landslide was anticipated on July 16, 1987, by weather forecasting which predicted exceptional perturbations in the following 72 hours. Such an information gave way, for the first time in Italy, to an important and complex informative chain among different technical and operating bodies of the Civil Protection. As a consequence, the Mayors of small villages located in the threatened area were induced to evacuate several settlements and to check the structures at risk (roads, bridges, banks of rivers, etc.). On July 18, rainfall was so intense that the Adda river overflowed, provoking floods and damage in numerous towns. In addition, the road network was practically out of work, and the employment of helicopters was necessary for rescue. Furthermore, about 500 volunteers were employed in the removal of mud from houses and in the management of shelter centres. During the following days, experts of different organizations (Lombardia Region, Fire Brigade, Italian Army etc.) started an accurate slope survey. On July 25, a very large crack was recognized on the Zandila Mt about 2 km over the Adda river, suggesting the probability that a huge landslide could be triggered. As a consequence, the Minister responsible for the Coordination of Civil Protection established in Val Pola the "Major Risks Committee" which declared the status of high landslide risk. Therefore, on July 27, the Minister decided the immediate evacuation of 1,280 people. Those fears came true on Tuesday, July 28, at 7:23, when 40,000,000 m$^3$ of rock and mud slipped towards Adda valley covering 2.5 km in 23 seconds (Figure 2b). The kinetic energy of the landslide was such that the landslide body raised about 300 m as a gigantic wave on the opposite slope of the valley, causing great damage. Thanks to the evacuation, only 27 people were killed, but 341 houses were completely destroyed and 1,545 were injured, then the timely evacuation highly
reduced the number of victims. The debris dammed the course of the Adda river, creating a dam and a lake, which grew at a constant speed of about 20 cm/hr up to a maximum height of 100 m. The risk of a dam break and of the consequent uncontrolled overflow of the lake induced the Ministry to entrust the Commission for the assessment of the risk and of the works to adopt for risk mitigation. Through different mathematically and physically based risk scenarios, the Commission decided to empty the lake through a pumping system. However, since, the Adda discharge was higher than the predicted on August 23, the lake level was already at the crest of the dam. Insofar, an anticipated artificially controlled overflow of the lake was rapidly conceived by digging a 10 m wide an 1 m deep channel on the crest of the landslide accumulation. The operations started on August 29, and were completed on August 31 (Figure 18), restoring the continuity of the Adda river about one month after the collapse. Further interventions included:

- strengthening of the landslide dam;
- construction of two by-pass tunnels;
- monitoring of the slope through the installation instruments capable to survey hydrometeorological, topographic and microseismic parameters.

According to a special Law (No. 102, 1990), the area involved in the event was classified as an “active landslide zone” and the prohibition of building was imposed.

In the years after, the National Department of Civil Protection has been involved in other events including the Sarno event in 1998. This caused 159 casualties in four villages (Sarno, Bracigliano, Siano and Quindici) located at the foot of the Pizzo d’Alvano mountain as a consequence of 40 different debris flows which were triggered in a time span of about 16 hours. In some towns (Quindici, Siano), risk areas were immediately evacuated. In fact, the phenomenon was immediately deemed critical, because a similar event, although of lesser intensity, had taken place in 1997 in Quindici, although in May 1998, there were some victims (11), a major tragedy was avoided. In contrast, in Sarno, the seriousness of the situation was underestimated so that there was no evacuation and more casualties were caused by debris flows. The Civil Protection intervened without delay but only in the aftermath of the event. More than 2 million m³ of mud were excavated, victims were recuperated and a young man was found alive under the mud three days after the event. Regular emergency plans were arranged: sheltering of the population, management of the aids coming from all over the country, organization of volunteering teams taking part in the excavation operations, safeguarding of the abandoned buildings in order to prevent robberies. Additional interventions took place such as:

- involvement, through the G. N. D.C. I., of the national scientific community which was involved in landslide analysis and in mapping of the areas still exposed to risk; the cartography was indeed produced in a very short time (Cascini, 2005);
- appointment of a Committee with the task to plan urgent measures for risk mitigation;
- appointment of a Field Survey Team with the task to survey and document the state of the sites involved in movements.
A key point was the question concerning the reconstruction of the most damaged and unsafe districts elsewhere, or plan their reconstruction on the same location, arranging adequate protection measures. The debate was animated even at the governmental level. Eventually, the hope to reconstruct the buildings on the same site prevailed, mainly because of the pressure of the population. In a few weeks, after the first reassessment operations had taken place, people slowly began to go back to the abandoned households. A very effective emergency plan was then arranged involving rainfall monitoring, establishment of pluviometric threshold values, institution of operative centres at each municipality, development of the activities of the Field Survey Team concerning direct site survey and recognizing of anomalous phenomena. The success of the early warning model adopted in Sarno (Picarelli et al. 2007a) has been taken as an example for the whole country. The reassessment plan has been effectual, including the realization of a number of works that have brought the risk to an acceptable level (Versace et al. 2007). In particular, precautionary measures have been adopted in the reconstruction phase, as the interdiction in the residential use of ground floors up to 6 meters, partitions in the ground floor unconnected from the structural frame, strengthening of this last to resist the pressure exerted by debris or even the blows caused by bodies dragged by the mud.

Even though the events of May 1998, uncovered the limits of the national system of civil protection, through the legislative measures which were then promulgated, they constitute a turning point for a further development in the defence against catastrophic landslides.
CONCLUSIONS

Italy is one of the parts of the world characterised by the highest risk of landslides. This is not only because of the high hazard posed by the geomorphological and geotechnical features of the territory, but mostly because of the density of population and of infrastructures all over the territory. Because of the number of disasters occurred, recently this situation became unbearable. As a consequence, some laws devoted to protection of the territory have been approved and different organizations have been set up for land management and planning and civil protection. This approach is supported by researchers and scientists which are giving a strong contribution not only in terms of knowledge improvement and spreading, but also in the operative approach.

Despite the present state of the art is complex and still unsatisfying, the increase in the knowledge of the mechanics of landslides and of the efficiency of the National Department for Civil Protection and of other institutions have determined a strong increase in the capability of the State, of the Regions and of the Municipalities to manage with some success such situations.

REFERENCES


Picarelli, L. (2001). “Transition from slide to earthflow, and the reverse.” *Proc. TC-11 Conference on Transition from slide to flow*, Trabzon, Turkey, on CD.


ACKNOWLEDGEMENTS
Prof. U. Maione, which provided data about the Valtellina emergency is warmly and gratefully acknowledged. Profs. Domenico Calcaterra and Antonio Santo contributed to the section on Historic Landslides in Italy. The Authors thank Dr Luca Comegna for his continuous support.
COUNTRY REPORT FROM JAPAN: PROGRESS OF LANDSLIDE DYNAMICS AND THE INTERNATIONAL PROGRAMME ON LANDSLIDES

Kyoji Sassa
International Consortium on Landslides
Kyoto University

Abstract: Landslide risk mitigation studies have gradually shifted from slope stability analysis to landslide risk assessment for land-use planning and early warning. Sustainable development and so many dangerous slopes cannot allow construction of expensive slope stabilisation works to mitigate landslide disasters in residential areas. One of the core research for landslide risk assessment is landslide dynamics. Landslide dynamics is the study focusing on the initiation mechanism by earthquake, rainfalls, and combined effects of both triggering factors, and the rapid post-failure motion. Recent progress in landslide dynamics, especially using the dynamic-loading ring shear apparatus are introduced in this paper.

Another report is the initiation and development of the International Programme on Landslides (IPL) since the foundation of the International Consortium on Landslides (ICL) in 2002 as an important global initiative to develop research to learn about landslide risk mitigation. This is not a research project itself, but an initiative to develop the background and infrastructure and funding of landslide studies. Joint efforts by international landslide research community are requested to develop this initiative.

INTRODUCTION

Sassa had a keynote lecture in the 6th International Symposium on Landslides on cyclic loading ring shear apparatus (DPRI-3) in 1992 (Sassa 1992; 1994). Thereafter, a series of dynamic loading ring shear apparatuses were developed up to DPRI-7. The initial and second apparatuses (DPRI-1 and 2) before DPRI-3 were high speed shearing at a constant speed under the drained condition. Stress-control to reproduce shear stress inside slope was not possible, and undrained condition and pore pressure monitoring were not possible.

After the Hyogo-ken Nambu earthquake in 1995, a much advanced apparatuses were developed obtaining special budget from the Government of Japan. The fifth and sixth apparatuses may reproduce stress inside slope during earthquakes or rainfalls or both effects under undrained condition or partially drained condition or drained condition. The maximum speed of shearing is 224 cm/sec for DPRI-6, and 300 cm/sec for DPRI-7 at the center of sample (Sassa et al. 2004). In this Landslide Disaster Forum, the following topics will be discussed: 1) undrained dynamic-loading ring shear test and its application to analyze the 2006 Leyte landslide, a rapid and long-travel landslide triggered by a combined effect of earthquake and rainfall, 2) computer simulation using the steady state shear resistance mobilized during post-failure motion which can be measured by the dynamic loading ring shear apparatus, 3) friction angle at steady state and friction angle at phase transformation, and 4) rate effect on the steady state friction angle in clayey sands.

The New International Consortium on Landslides were founded to establish the International Programme on Landslides (IPL) by the group of the International Geological Correlation...
Programme (IGCP) No. 425 “Landslide Hazard Assessment and Cultural Heritage” in 2002. IGCP programme is a joint initiative of UNESCO and IUGS (International Union of Geological Science). Allocated budget was minimum, but the authorization as an International Programme could greatly support 31 sub-group project leaders of IGCP-425. Each project will be terminated within 5 years. Participants were willing to found a New International Landslide Programme (IPL) by the international joint efforts of landslide community. These joint efforts were supported by the United Nations Organizations of UNESCO, WMO, FAO and UN/ISDR. IPL was developed and the First World Landslide Forum in November 2008 at the United Nations University, Tokyo to implement the 2006 Tokyo Action Plan on the International Programme on Landslides. The forum is neither a pure scientific/engineering meeting, nor a pure governmental and intergovernmental meeting, but a meeting cross-cutting many fields which may contribute to landslide and other related earth system disaster reduction.

PROGRESS OF LANDSLIDE DYNAMICS IN JAPAN

Development of a Dynamic Loading Ring Shear Apparatus and Computer Simulation

The ring shear apparatus was developed to obtain the residual shear resistance under a large shear displacement in landslides. The most well-known and widely adopted type of ring shear apparatus was developed jointly by scientists and engineers at the Imperial College of Science and Technology (United Kingdom) and the Norwegian Geotechnical Institute (Bishop et al. 1971; Bromhead 1979; 1986). Other types of ring shear apparatuses were developed, but the basic purpose is to investigate the residual shear resistance under a large shear displacement as a parameter of soils. The shearing in all ring shear apparatuses was provided by speed-control.

On the contrary, the purpose of DPRI ring shear apparatus is to quantitatively simulate the entire process of failure of a soil sample, from initial static or dynamic loading, through shear failure, pore-pressure changes and possible liquefaction, to large-displacement, steady-state shear movement. Natural phenomena of shearing within slopes is NOT the speed control condition, but the stress control phenomena. As shown in Figure 1, sample is collected from the slope in which a sliding surface was or will be formed. The sample is set in the apparatus, and the stress and water condition before landslides are reproduced by supplying water, consolidating under normal stress first, then loading under shear stress. Thereafter, triggering stresses are loaded, such as pore pressure increase, earthquake loading, a combined stresses of both, or other stress change induced by toe erosion, excavation, banking, or dynamic loading by a landslide mass from the upper slope. Phenomena resulted from such stress loading are observed; whether shear deformation will result in shear failure, whether sliding surface will be formed, how much pore pressure will be generated within the soil sample, whether rapid and long shear displacement after failure will be caused or only limited slow displacement will occur. The shear resistance mobilized before and after failure and in the steady state are monitored. The basic concept is NOT shear strength test, but a physical simulation test to reproduce the shearing phenomena in slopes. Landslide phenomena are complicated and affected by many factors, and there are many unknown factors and processes. Therefore, this approach will be reasonable, especially for the matter of dynamic loading. The detailed structure and method are explained in Sassa et al. (2004).
Figure 1: Concept of the stress-controlled dynamic loading ring shear test to geotechnically simulate the formation of sliding surface and the resulting post-failure motion.

Application of this Dynamic Loading Ring Shear Apparatus to the Leyte Landslide 2006 Induced by a Combined Effect of Rail Fall and a Nearby Small Earthquake
A rapid and long-traveling landslide occurred on 17 February 2006 in the southern part of Leyte Island, Philippines (Catane et al. 2007). The landslide resulted in 154 fatalities and 990 people disappeared in the debris. A Japanese and Philippine joint team investigated the site and took samples from the landslide. From field investigation and dynamic-loading ring-shear tests on a sample taken from the landslide site, we report here that: 1) a small (Ms 2.6), near-by earthquake was strong enough to trigger the landslide which occurred after heavy rainfall. Thus, this landslide was rainfall and earthquake induced, and 2) the subsequent rapid motion of the landslide was a consequence of the “sliding-surface liquefaction phenomenon”. It was the result of the generation of high porewater pressure within the shear zone, which was caused by crushing of grains of the volcanoclastic debris at the site subjected to shearing under a high normal stress.

Figure 2 is a front view of the landslide. The slope had been covered by tropical forest. The landslide removed the forest-cover. Three linear concave ground forms (F1, F2, and F3) were identified by observation from the ground and also from the helicopter. These concave ground forms were the result of fracturing of rocks due to the Philippine fault and its sub-faults. Along one of these concave ground forms (F3), a secondary landslide, which occurred after the main landslide, was found at point A in Figure 2. The occurrence of the landslide suggests lower shear strength along these fractured zones. The central line of the landslide seemed to bend at B and C, which is likely a combination of two parallel lines (Figure 2).
Figure 2: The location of the 2006 Letyte landslide and a front view from a helicopter (photo by K. Sassa)

The section of the central line of the landslide was surveyed by a non-mirror total station and a ground-based laser scanner in the field and compared to a SRTM (Shuttle Radar Topography Mission) map before the landslide (Figure 3). The red-color part shows the initial landslide mass while the blue-color part presents the displaced landslide debris after deposition. The initial landslide mass shown in red seems to consist of two blocks (Block 1 and Block 2) corresponding to the two straight lines above/below the bending part in Figure 2. Though exact verification is difficult, we can speculate a landslide development process from the photo and section as well as the field observation: 1) The groundwater level rose upslope from point B in Figure 2 because the ground water flow probably was blocked at this bending point and likely dammed up upslope of this point; 2) This blockage necessarily lowered the stability of the lower part of Block 1 in Figure 3; 3) The loss of stability in the lower part of
Block 1 should have almost simultaneously caused the motion of the rest of Block 1, including the head scarp due to loss of support at its bottom, and, at the same time, the movement of Block 1 provided additional forces onto the top of Block 2 initiating the movement of Block 2; and 4) Both landslide masses traveled onto the flat residential and farming areas. The inclination connecting the top of the initial landslide and the toe of the displaced landslide deposit is approximately 10°, which indicates the average apparent friction angle mobilized during the whole travel distance. This value is much smaller than the usual friction angle of debris (sandy gravel) of 30° - 40°. Therefore, it suggests that high excess pore-water pressure was generated during motion. Figure 3(b) shows a flow mound that traveled from the initial slope to this flat area without much disturbance. Movement without much disturbance is possible when the shear resistance on the sliding surface is very low; thus, movement of the material is like that of a sled.

![Figure 3](image)

Figure 3: (a) Central section of the landslide and (b) a flow mound at the sampling point. The topography of the surface after landslide was measured by non-mirror total station and ground-based laser scanner. The ground surface before landslide is obtained by SRTM (Shuttle Radar Topography Mission)

The material of the flow mound is volcanoclastic debris, including sand and gravel. Though the sizes of flow mounds on this landslide were diverse, most of them consisted of volcanoclastic debris. We also observed from the surface and excavation that the valley-side slope in the source area consisted of volcanoclastic debris or strongly weathered volcanoclastic rocks. Therefore, we took a sample of about 100 kg from the base of the flow
mound shown in Figure 3 (S in Figure 2). We subjected this sample to dynamic-loading ring-shear tests assuming the sample to be the soil in which the sliding surface was formed.

We examined the triggering factors of the Leyte landslide. A small earthquake occurred near the site at the time of occurrence of the landslide, and the location of the hypocenter of this earthquake was estimated by the U.S. Geological Survey (USGS) and the Philippine Institute of Volcanology and Seismology (PHIVOLCS). According to PHIVOLCS, the earthquake occurred at a location 21 km west of the landslide, 8 km deep, with a magnitude of Ms 2.6, at 10:36 hrs on 17 February 2006. Using the standard attenuation function between peak ground acceleration and hypocentral distance, the peak ground acceleration at the landslide site was estimated at 10 gal for this magnitude. We then estimated the expected peak acceleration at the bottom of the landslide mass as about 60 gal. This was based on about three times ground accelerations at the sliding surface due to the difference in the compressional (P) wave speeds between soft volcanoclastic debris (Vp= 0.5 – 1.5 km/s) and hard volcanic bedrock (Vp= 2.5 – 5 km/s) that outcropped in the head scarp, because the amplification level is proportional to the velocity contrast between two layers. Though the shear (S) wave speeds of the volcanoclastic debris and the bedrock are unclear, similar level of velocity contrast to the P wave is expected at the sediment/bedrock interface. An additional magnification of two times is also expected in the landslide site due to the focusing of seismic waves on the steep mountain topography; this resulted in a total magnification of six times.

Heavy rainfall (459.2 mm for 3 days on 10-12 February and 571.2 mm for 5 days on 8-12 February 2006) occurred in this area before the day of the landslide. This rainfall should have increased the ground-water level and pore-water pressure inside the slope. However, the peak ground-water level had likely passed before the occurrence of the landslide on 17 February because the rainfall on 13-17 February was small (a total of 99.0 mm for 5 days). We deduced that a small earthquake was the trigger of the landslide (more details are described in Sassa et al. 2007).

The sample was set in the shear box (250 mm inside diameter and 350 mm outside diameter) of DPRI-6, was fully saturated, and was consolidated under the stresses acting on the sliding surface in the lower part of Block 1. From Figure 3, it can be seen that the depth of the sliding surface at the lower part of Block 1 is 50-70 m. However, a lesser soil depth was assumed because of the capacity of this apparatus: the sliding surface was assumed for the test to be 35 m deep and at an inclination of 25°. The unit weight of the soil was assumed to be 20 kN/m³. In the preliminary test to increase porewater pressure until failure, the critical ground-water level causing a landslide without earthquake loading in this slope was obtained. In the simulation test of a rainfall and earthquake-induced landslide, the normal stress corresponding to that of 5 m lower than the critical groundwater level (i.e., further rise of groundwater level by 5 m will trigger the landslide) was first loaded on the sample in the ring-shear apparatus and consolidated. Then, the shear stress due to the self-weight of the soil layer was loaded. Finally, a peak seismic stress corresponding to a seismic acceleration of 60 gal using the ground acceleration observed in Maasin (PHIVOLCS, Code number: MSLP, Latitude: 10.1340, Longitude: 124.8590, Elevation: 50.0) was loaded on the sample.
Figure 4: Test result to geotechnically simulate initiation of the 2006 Leyte Landslides triggered by a nearby small earthquake after rainfall

The time series data of monitored parameters are presented in Figure 4(a). The black line shows the normal stress loaded on the sample, the red line shows the shear resistance mobilized at the sliding surface; the blue line shows the monitored pore-pressure change inside the sample; and the shear displacement at the center (300 mm diameter) of the ring-shaped sample is shown in green. With progress of shearing, the porewater pressure increased while the shear resistance decreased. Then, the pore pressure reached close to the normal stress and a steady-state of high-speed shearing proceeded at a very low shear resistance of only 7 kPa. The stress path of this test is shown in Figure 4(b). It visualizes that a small seismic loading caused shear failure which was followed by a rapid stress reduction due to the “sliding surface liquefaction” described in Sassa (1996) and Sassa et al. (2004).

This study of the Leyte landslide showed that even a small earthquake can be the critical trigger of a landslide when the stability of the slope has already been reduced due to rainfall, and demonstrated that the complicated dynamic landslide phenomenon can be studied and the capability of landslide hazard assessment by the recent geotechnical technology.
**Computer Simulation using the Steady State Shear Resistance Mobilized during Post-Failure Motion which can be Measured by the Dynamic Loading Ring Shear Apparatus**

The comparison of various existing landslide mobility modelling is conducted on 12 December in this Landslide Disaster Forum. Sassa proposed a computer simulation concept and programme as a special lecture in the 5th International Symposium on Landslides in 1988. The programme was modified to reflect the progress of knowledge of steady state shear resistance obtained by the undrained ring shear apparatus. Furthermore, the programme was professionally improved by the budget of IPL-M101 APERITIF Project -Arial Prediction of Earthquake and Rain-Induced Rapid and Long Traveling Flow Phenomena- by the Special Coordinating Fund for Promoting Science and Technology of the Government of Japan (Representative: K. Sassa) (Sassa 2004), and developed into a user-friendly programme. The detailed application is reported by Wang & Sassa (2007) in this forum.

Sassa (1988) proposed a geotechnical model for the motion of landslides as illustrated in Figure 5. It is a quasi-3D frictional model. The first condition used for this simulation is the equilibrium of forces acting on a vertical soil column. The component parallel to slope of gravity force ($W_p$) + Lateral stress + Shear Resistance ($R$) should be the same with mass of the column ($m$) times acceleration ($a$).

\[
am = W_p + \frac{\partial P_x}{\partial x} + \frac{\partial P_y}{\partial y} + R \tag{1}
\]

The second condition is the landslide volume is kept constant during motion, such that the volume flowing into the column is the same with the increase of landslide volume inside the column due to height change.

The two basic geotechnical parameters used in this simulation are:

1. Apparent friction angle $\phi_a$ which is the angle of line connecting from the origin to the stress given by the mobilized shear resistance (shear stress at steady state) and the total normal stress. The apparent friction angle in the steady state is expressed as,

\[
\tan \phi_a = \frac{\tau_s}{\sigma} \tag{2}
\]

This parameter controls mobility of landslide mass.

2. Lateral pressure ratio $k$ (ratio between lateral stress and vertical stress) which represents the softness of landslide mass.

\[
k = \frac{\sigma_h}{\sigma_v} \tag{3}
\]

where $k=1.0$ means completely soft like liquid and $k=0$ means very hard like a rock mass. The lateral stress ratio is estimated by the Jarky’s equation ($k = 1 - \sin \phi_a$), where $\phi_a$ is the apparent friction angle inside the soil mass.
Figure 5: Concept of the computer simulation based on the geotechnical model for the motion of landslides (Sassa 1988) and its modification (Wang & Sassa 2002)

Figure 6: Effective stress paths at different initial normal stresses on the Osaka formation sandy soils. The void ratios after consolidation for the four tests are the same as 0.71 (Okada et al. 2000)

The most important aspect of this simulation method was that the author tried to measure or estimate geotechnical shear characteristics ($\phi_a$ and $k$) of soil samples and apply those values into the computer simulation. Most of computer simulation techniques were originally proposed in other fields, so geotechnically measured parameters are not always used in the simulation. If the key parameters input in the computer simulation cannot be measured for the site in consideration, the simulation is a “back analysis” to find good values of parameters. Therefore, they cannot be used for “prediction” even if a detailed investigation will be conducted before landslides.

The measurement technology of geotechnical parameters to simulate the motion of landslides has progressed by the development of undrained dynamic loading ring shear apparatus (Sassa 1994; Sassa et al. 2004). Students in the Disaster Prevention Research Institute of Kyoto University implemented many tests. Some of test results are shown in Figures 6, 8, 12 and 13. Many of sandy soils which are subjected to grain crushing and the resulting volume reduction
often reached a low steady state at a certain normal stress under which further grain crushing and resulting volume reduction did not occur any more. Okada et al.2000 tested a sandy soil sample at same void ratio in different normal stress (Figure 6). From tests at different densities reached a similar steady state value of 12-24 kPa (Figure 8). We assumed that the steady state value could be regarded to be kept constant during the motion of landslide independent of thickness of landslide mass. In this case, the apparent friction angle mobilised during motion will be changed due to the thickness of landslide mass, namely normal stress. As shown in the right bottom of Figure 5, the reduction of thickness (normal stress) increases the mobilised apparent friction angle. When a landslide mass moves onto a flat area, the mass is spread and the thickness is decreased, then the apparent friction angle increases, then the landslide mass decelerates and stops.

The effects of two parameters are shown in Figure 7. The travel distance will increase inversely with the value of steady state shear strength, namely the apparent friction angle from the relationship of \((\tan\phi_s = \tau_{ss}/\sigma)\). The lateral pressure ratio affects the deformability and softness of landslide mass, and the height of landslide mass. The application of this simulation is presented in this forum (Wang & Sassa 2007).

<table>
<thead>
<tr>
<th>(k)</th>
<th>(\tau_{ss} = 20) kPa</th>
<th>(\tau_{ss} = 30) kPa</th>
<th>(\tau_{ss} = 50) kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td><img src="image1" alt="Diagram" /></td>
<td><img src="image2" alt="Diagram" /></td>
<td><img src="image3" alt="Diagram" /></td>
</tr>
<tr>
<td>0.6</td>
<td><img src="image4" alt="Diagram" /></td>
<td><img src="image5" alt="Diagram" /></td>
<td><img src="image6" alt="Diagram" /></td>
</tr>
<tr>
<td>0.9</td>
<td><img src="image7" alt="Diagram" /></td>
<td><img src="image8" alt="Diagram" /></td>
<td><img src="image9" alt="Diagram" /></td>
</tr>
</tbody>
</table>

Figure 7: Effects of the steady state shear resistance and the lateral pressure ratio on the movement of landslides
Recent Progress on the Steady State Shear Resistance based on Undrained Ring Shear Apparatus

**Friction Angle at Steady State and Friction Angle at Phase Transformation**

The friction angle at steady state is important for landslide dynamics which can be measured by the undrained ring shear apparatus. When this value can be estimated from conventional shear tests, it will be more convenient.

Undrained ring shear tests were conducted investigating on the effects of gradation of grains. Various grain size distributions of sands were produced by mixing various grains of quartz sands. Different from the original purpose, some interesting results are obtained. Those test results are shown in Figures 8 to 10 (Igwe et al. 2007). Two conclusions can be obtained from these results:

1. As easily found from Figure 10, the effective friction angle at steady state is approximately the same as the angle of phase transformation line. It can be measured from the constant volume shear box test, or the triaxial test. Both values are independent of relative density, namely material constant.

2. The ratio between generated pore pressure and shear stress increment during shear stress loading divides the post failure behavior, “collapse” directly going to the steady state before reaching the failure line, or “dilative” dilating after passing the phase transformation line. Just between two cases, there is a state (threshold) at which friction angle of steady state, friction angle at peak, and angle of phase transformation.

![Diagrams](Figure 8: Friction angle at steady state and the angle of phase transformation line in the medium and dense state (Igwe et al. 2007))
Figure 9: Relationship between normal stress and the ratio of the increments of generated pore pressure and loaded shear stress

Figure 10: The range of friction angles at steady state and at threshold, and the angle of phase transformation in different relative densities (WG: well graded, ING: intermediately graded, NG: narrowly graded, GAG: gap graded)
Rate Effect on the Steady State Friction Angle in Clayey Sands
Saito et al. (2006) investigated the rate effect of clayey sands using undrained ring shear test at the speed (rate) control condition. The test programme is presented in Figure 11. A clay-sand mixture is fully saturated and consolidated and sheared under a constant rate (Stage 1), after around 1 m shearing, the shear rate was changed (Stage 2), it was continued until Stage 8. Through the stages, the total normal stress was kept constant at 100kPa, and generated pore pressure was continually monitored. Then, effective stress path and effective friction angle at steady state were observed for each stage. Used samples are sands including 0, 10% or 20 % of clays (Bentonite, Illite, Kaoline).

Figure 11: Test procedure showing the change in shearing rate from Stage 1 to Stage 8

The test results are presented in Figure 12 and Figure 13. Figure 12 is the effective stress paths obtained from tests on silica sands without clay (Figure 12(a)) and silica sand-bentonite (20%) mixture (Figure 12(b)). For silica sands, effective stress paths at all stages ride on the same failure line of 34.1 degrees. While for sand-bentonite (20%) mixture, two distinct failure lines are obtained, one for slow shearing rate of 0.1mm/sec (32°), one for greater shearing rate of 1.0 mm/sec (14.0°). Clear effects of shearing rate were found. The same procedure of tests were repeated.

Figure 12: Effective stress paths at different shear rates (a) Silica sands; (b) Bentonite (20%) - Sand mixture Shear rate of 1 mm/sec
Figure 13 presents all test results; no rate effects were found for sands, and sand-korine mixture (10% and 20%) and sand-illite mixture (10%). Slight rate effects appeared in 10% mixture of bentonite and 20% mixture of illite. The greatest effect was observed for 20% bentonite mixture in the saturated condition. As a check, dry 20% dry bentonite mixture was also tested (Figure 13(e)). A minimum rate effect was found. The reason of lower friction angle is likely to be the arrangement of platy particles parallel to the shear surface in greater shearing rate. Further studies are conducted by Saito.

Note
SS: sand only (no clay)
B-10, B-20, I-10, I-20, K-10, K-20: saturated state of 10%, 20% mixture of Bentonite, Illite, Kaorin clays
B-20D: dry state of B-20 clay sand mixture

Figure 13: Effective friction angles of different clay-sand mixture

INTERNATIONAL PROGRAMME ON LANDSLIDES
The International Consortium on Landslides (ICL), created during the Kyoto Symposium in January 2002, is an international non-governmental and non-profit scientific organization, which is supported by the United Nations Educational, Scientific and Cultural Organization (UNESCO), the World Meteorological Organization (WMO), the Food and Agriculture Organization of the United Nations (FAO), the United Nations International Strategy for Disaster Reduction (UN/ISDR), and intergovernmental programmes such as the International Hydrological Programme of UNESCO; the International Union of Geological Sciences (IUGS); the Ministry of Education, Culture, Sports, Science and Technology (MEXT) of the Government of Japan, U.S. Geological Survey, and other governmental bodies. ICL was
registered as a legal body under Japanese law for non-profit organizations in August 2002 in the Government of Kyoto Prefecture, Japan. The present domicile of the International Consortium on Landslides is Kyoto, Japan, where the secretariat is located. The objectives of the consortium are to:

1. promote landslide research for the benefit of society and the environment, and capacity building, including education, notably in developing countries;
2. integrate geosciences and technology within the appropriate cultural and social contexts in order to evaluate landslide risk in urban, rural and developing areas including cultural and natural heritage sites, as well as contribute to the protection of the natural environment and sites of high societal value;
3. combine and coordinate international expertise in landslide risk assessment and mitigation studies, thereby resulting in an effective international organization which will act as a partner in various international and national projects; and
4. promote a global, multidisciplinary programme on landslides.

The central activity is the International Programme on Landslides (IPL). Other activities planned include international co-ordination, exchange of information and dissemination of research activities and capacity building through various meetings, dispatch of experts, landslide database, and publication of *Landslides*: Journal of the International Consortium on Landslides. The outline of ICL and the relationship of ICL, IPL and Research Centre on Landslides (RCL) are illustrated in Figure 14.

![Framework for ICL, IPL, and RCL](image_url)

Figure 14: Cooperation structure of ICL and IPL for global landslide risk reduction

ICL members come from one of four categories of intergovernmental organizations, non-governmental organizations, governmental and public organizations, and other organizations and entities. ICL is supported by the United Nations Educational, Scientific and Cultural Organization (UNESCO) / the World Meteorological Organization (WMO) / the Food and Agriculture Organization of the United Nations (FAO) / the United Nations International Strategy for Disaster Reduction Secretariat (UN/ISDR) / the United Nations University (UNU) / International Union of Geological Sciences (IUGS) / Governments of Japan, USA, Italy, Canada, and Norway. The Research Centre on Landslides was established in the Disaster Prevention Research Institute, Kyoto University, Japan in 2003. The European Center of the International Consortium was established in the University of Florence, Italy in 2006.
The International Consortium on Landslides aims to promote international cooperation between developed countries and developing countries. The current member organizations are both from developed countries and developing countries.

The International Program on Landslides (IPL) was initiated at the Board of Representatives Meeting held at UNESCO in 2002. It aims to conduct international cooperative research and capacity building on landslide risk mitigation, notably in developing countries. Activities of IPL, through its various projects, contribute to the International Strategy for Disaster Reduction (ISDR). IPL projects initiated by ICL with supports from UNESCO, WMO, FAO, UN/ISDR, UNU, IUGS and others.

The 2006 Tokyo Round Table Discussion “Strengthening Research and Learning on Earth System Risk Analysis and Sustainable Disaster Management within UN-ISDR as Regards Landslides” towards a dynamic global network of the International Programme on Landslides (IPL) was held at the United Nations University, Tokyo, in 2006. As the result of this meeting, the 2006 Tokyo Action Plan towards a global dynamic network of IPL was adopted.

Based on the 2006 Tokyo Action Plan towards a dynamic global network of International Programme on Landslides, this programme (IPL) has developed to a new global joint programme between ICL, UNESCO, WMO, UN/ISDR, UNU, ICSU, WFEO which has exchanged MoU with ICL for the promotion of IPL for global landslide risk reduction.

New global IPL projects will be proposed and approved in the Second IPL Global Promotion Committee (Chairperson: Salvano Briceno, Director of United Nations Secretariat for the International Strategy for Disaster Risk Reduction) to be held at the United Nations University in January 2008. IPL projects have two categories of coordinating project (long term period and proposed by IPL Global Promotion Committee, and member project proposed by one or more member organizations and approved by the IPL Global Promotion Committee. New global IPL project will include currently on-going projects; C100 Landslides: Journal of the International Consortium on Landslides, C101 Landslide risk evaluation and mitigation in cultural and natural heritage sites, C103 Global landslide observation strategy, C104 World Landslide Database, a new project of “Early Warning of Landslides” proposed on the new project by six Asian countries. Member projects will include currently active member projects and newly proposed projects. As a global platform to examine the development of International Programme on Landslides strengthening research and learning on earth system analysis and sustainable disaster management within UN-ISDR as regards “Landslides”, the First World Landslide Forum will be organized in November 2008 (see website: http://iclhq.org).

2006 TOKYO ACTION PLAN ON LANDSLIDES AND RELATED EARTH SYSTEM DISASTERS
The World Conference on Disaster Reduction (WCDR) was held on 18–22 January in Kobe, Japan in which a session titled “New International Initiatives for Research and Risk Mitigation of Floods (IFI) and Landslides (IPL)” was organized by ICL, United Nations Educational, Scientific and Cultural Organization (UNESCO), the World Meteorological Organization (WMO), the Food and Agriculture Organization of the United Nations (FAO), the Ministry of Education, Culture, Sports, Science and Technology of the Government of Japan (MEXT), the United Nations University (UNU), Kyoto University (KU), and others. Within the session, a Letter of Intent to promote further joint global activities in disaster
reduction and risk prevention through “Strengthening research and learning on ‘Earth System Risk Analysis and Sustainable Disaster Management’ within the framework of the ‘United Nations International Strategy for Disaster Risk Reduction’ (ISDR)” was proposed by ICL, and approved and signed within 2005 by Director-Generals or Secretary-Generals of UNESCO, WMO, FAO, UN/ISDR, UNU, ICSU, WFEO. The Letter of Intent can be an umbrella for all initiatives of earth system risk reduction. It was approved and signed by seven global stakeholders.

Based on this Letter of Intent, a round table discussion was organized at the United Nation University in Tokyo on 18–20 January 2006. The meeting aimed to examine the global plan promoting the research and learning on landslides and developing the International Programme on Landslides as a dynamic global network. The round table discussion was jointly organized by the International Consortium on Landslides (ICL), United Nations Educational, Scientific and Cultural Organization (UNESCO), World Meteorological Organization (WMO), Food and Agriculture Organization of the United Nations (FAO), United Nations International Strategy for Disaster Risk Reduction (UN/ISDR), United Nations Environment Programme (UNEP), United Nations University (UNU), Kyoto University (KU).

As the result of discussion, participants adopted the “2006 Tokyo Action Plan Strengthening Research and Learning on Landslides and Related Earth System Disasters for Global Risk Preparedness.” The 2006 Tokyo Action Plan is an important milestone in the global landslide community for its development in the future (see Appendix A).

CONCLUSION
Landslides are a natural phenomenon often combined with other hazards due to heavy rain falls, earthquakes, volcanic eruption, Tsunami, and human impact. The magnitude of landslide disaster was underestimated because one event is not always great, but the number of landslides are so many. Landslides are involved in many fields of science, many of government ministries. However, landslides are not always be the central issue of those scientific fields and those ministries because of the multi-disciplinary characteristics. There was no international programme dealing with landslides though it may be a part of International Geoscience Programme (IGCP), International Hydrological Programme (IHP). The understanding of the multi-disciplinary and cross-cutting characters of landslides has been gradually developed. It reaches the plan of the First World Landslide Forum.

The First World Landslide Forum is a global cooperation platform for all types of organizations and individuals. It is neither a pure scientific/engineering meeting, nor a pure governmental and intergovernmental meeting, but a meeting cross-cutting many fields which may contribute to landslide and other related earth system disaster reduction. We request participants of this 2001 International Forum on Landslide Disaster Management to present the most advanced science, technology and disaster management in this global cooperation platform and also contribute to create stronger infrastructure for landslide risk mitigation studies.

REFERENCES
Geotechnique, 21(1), 273–328.

APPENDICES
Appendix A – 2006 Tokyo Action Plan
APPENDIX A
2006 TOKYO ACTION PLAN

Strengthening Research and Learning on Landslides and Related Earth System Disasters for Global Risk Preparedness (Adopted in the Round Table Discussion on 20 January 2006 in Elizabeth Rose Hall of the United Nations University, Tokyo)

The 2006 Tokyo Round Table Discussion “Strengthening Research and Learning on Earth System Risk Analysis and Sustainable Disaster Management within UN-ISDR as Regards Landslides” towards a dynamic global network of the International Programme on Landslides (IPL) was held at the United Nations University, Tokyo, from 18th to 20th January, 2006 to formulate a framework for cooperation and to identify focus areas to reduce landslide risk worldwide. The following action plan was adopted as a summary of the meeting, to be implemented within the scope of the Hyogo Framework for Action 2005-2015, “Building the Resilience of Nations and Communities to Disasters”, declared at the United Nations World Conference on Disaster Reduction held in Kobe, Japan in 2005.

Preamble
Large and small landslides occur almost every year in nearly all regions of the world. Figure A1 shows the example for casualties in Japan for 1967-2004. Landslide disasters in Japan have occurred every year; the total number of deaths due to landslides is about one half of those caused by earthquakes, including the catastrophic 1995 Kobe earthquake.

Figure A1: Comparison of the numbers of victims in Japan from 1967-2004 due to landslide disasters, earthquake disasters including deaths by earthquake-induced landslides, and volcanic disasters including deaths due to volcanic gas (The statistic of victims by landslide disasters since 1967 was published by the Sabo Technical Center)

“Landslides” are a complex-disaster phenomenon that can be caused by earthquakes, volcanic eruptions, heavy rainfall (typhoons, hurricanes), sustained rainfall, heavy snowmelt, unregulated anthropogenic developments, mining, and others (Figure A2(a)). Large-scale
coastal or marine landslides are known to cause tsunami waves that kill many people; an example was the 1792 UNZEN-Mayuyama landslide, which caused a devastating tsunami that resulted in 16,000 fatalities from the landslides and the tsunami in Japan. Also large-scale landslides on volcanoes can dislocate the mountain tops and trigger volcanic eruptions; such was the case for the 1980 eruption of Mount St. Helens in the USA and presumably for Mt. Bandai in Japan. Landslides also may occur without earthquakes, heavy rains, volcanic eruptions, or human activities, due to progress of natural weathering; therefore, they occur almost everywhere in the world. Landslides most commonly impact residents living on and around slopes.

Landslides are a natural phenomenon which can only be effectively studied in an integrated, multi-disciplinary fashion, including contribution from different natural and engineering sciences (earth and water sciences), and different social sciences. This is also the case because landslides are strongly related to cultural heritage and the environment (Figure A2(b)). Landslides should be jointly managed by cooperation of different ministries and departments of government including some representing education, science and technology, construction and transportation, agriculture, forestry, and the environment, culture and vulnerable groups (the poor, aged, handicapped, or children). As landslides are highly localized phenomena, it is crucial to seek the contribution of local governments or autonomous communities (Figure A2(c)).

**Figure A2:** Characteristics of landslide disasters

The disasters caused by landslides are of very complex nature wherever they occur around the world. Research on landslides should be integrated into a new multi-disciplinary science field of landslide study. Landslide risk preparedness is to be managed by multi-ministries.

**Action Plan**
Global cooperation in landslide-risk reduction research and learning will be carried out encompassing related disasters affecting the earth-system, such as heavy rainfall, earthquakes, volcanic eruptions, tsunamis, and disasters of anthropogenic origin. Establishment of a ‘Dynamic Global Network of the International Programme on Landslides’ and its operation will effectively function for landslide and related risk reduction through the implementation of the following Action Items adopting a multi-hazard, multi-sectoral approach.
Actions

1. Establishment of the IPL Framework

   a) Establishment of the IPL Global Promotion Committee
   The IPL Global Promotion Committee shall be established by ICL members and ICL supporting organizations, as illustrated in Figure A3. The committee will meet annually, on the occasion of ICL Board of Representative meetings, or possibly at other occasions and locations. The committee will conceive a strategy to promote the 2006 Tokyo Action Plan, and will discuss the management of IPL global cooperation fields, and their possible modification, selection, and termination.

   b) Establishment of IPL World Centre
   The IPL World Centre will be established to coordinate and support implementation of the global cooperating fields of the International Programme on Landslides (IPL), which works as the secretariat of the IPL Global Promotion Committee and the International Programme on Landslides (IPL). The Centre will be hosted by the Headquarter of the UNESCO-KU-ICL UNITWIN Cooperation Programme “Landslide Risk Mitigation for Society and the Environment” in the Research Centre on Landslides, Disaster Prevention Research Institute, Kyoto University, Kyoto, Japan, where the secretariat of the International Programme on Landslides has been located since its foundation in 2002.

2. Promotion of the Global Cooperating Fields of the International Programme on Landslides (IPL)

   The global cooperating fields of IPL are identified as follows for the initial phase:

   a) Technology Development
      (i) Monitoring and Early Warning
         - Use of various on-site, in-situ technologies, as well as satellite observations in monitoring landslide effects and contributing factors for
early warning purposes
- Development of automated monitoring methods covering large spatial extent and real-time data communication, as well as low-cost monitoring devices
- Development of early warning methodologies, in particular for rain-induced landslides
- Applications linking meteorological, hydrological and landslide models

(ii) Hazard Mapping, Vulnerability and Risk Assessment
- Hazard Mapping at local and global scales
- Vulnerability assessment, considering human life, land resources, structures, infrastructure, and cultural heritage
- Risk assessment and communicating risk in an easily understood manner

b) Targeted Landslides: Mechanisms and Impacts
(i) Catastrophic Landslides
- Catastrophic landslides induced by natural and anthropogenic factors such as rainfall, earthquakes, volcanic activity, river erosion, and human activities, and their combinations
- Landslides threatening human lives and high societal values
- Gigantic coastal landslides and marine landslides causing tsunamis

(ii) Landslides Threatening Heritage Sites
- Studies for protection of cultural heritage, cultural landscape, and the natural heritage from landslides using non-invasive technologies and appropriate mitigation strategies (e.g. Machu Picchu, Bamiyan, Lishan, Cordillera Blanca)

c) Capacity Building
(i) Enhancing Human and Institutional Capacities
- Building human capacities and expertise in landslide management
- Institution building at national and local levels through Centers of Excellence
- Enhancing implementation and action at local level

(ii) Collating and Disseminating Information/ Knowledge
- Developing a culture of awareness on landslide risks
- Developing model policy frameworks, standards, guidelines/checklists, and training modules.

d) Mitigation, Preparedness and Recovery
(i) Preparedness
- Strengthening disaster preparedness of all stakeholders
- Strengthening capacities of communities and local institutions to cope with landslide hazards
- Forecasting and providing early warning of adverse conditions likely to lead to landslide activity
- Preparing contingency recovery plans, including pre-positioning of technical and material resources for likely landslide events

(ii) Mitigation
- Development of innovative, low-cost, and ecologically appropriate landslide mitigation techniques.
- Mountain conservation methods, including soil conservation, forest and
watershed management, and appropriate land-use techniques
- Appropriate civil engineering works, including construction and urban and coastal development;
- Restricting inappropriate development in landslide prone areas
- Development of appropriate policy and planning mechanisms, such as land-use management (including zoning)
- Promotion and strengthening of monitoring and warning systems

(iii) Recovery
- Post-landslide recovery and rebuilding efforts should integrate landslide mitigation measures
- Prevention of secondary risks of landslides resulting from inappropriate re-building efforts in response to any disaster (for example, earthquakes, volcanic eruptions, extreme weather events, etc.)
- Implementation of landslide recovery efforts and programmes (including psycho-social and health aspects) with the participation of affected communities and local authorities
- Providing long-term support to ensure sustainable recovery

3. Promotional Activities

a) World Landslides Forum
Capitalizing on the competence, international experience and established organizational network of ICL-IPL, it is proposed to create a global information platform for future joint activities of the worldwide landslide community, named the ‘World Landslide Forum’ that shall be convened every 3 years.

The first World Landslides Forum, organized by the ICL, can be planned to take place in January 2009, bringing together academics, practitioners, politicians, etc. to a global, multidisciplinary, problem-focused platform. This forum will provide an opportunity for the first identification of a WCoE. Linkages to ISDR activities, as well as other global events, including the World Water Forum, the International Year of Planet Earth, etc., will be established.

b) Identification and Promotion of World Centres of Excellence on Landslide Risk Reduction
The IPL Global Promotion Committee will identify and promote World Centres of Excellence (WCoE) every 3 years within eligible organizations, such as universities, institutes, NGOs, government ministries and local governments, contributing to “Risk Reduction for Landslides and Related Earth System Disasters”. Linkages to WCoE at the national level will be used to promote cooperation with the ICL and dissemination of knowledge and information. An independent panel of experts, set up by the Global Promotion Committee of IPL, may be appointed to endorse the WCoEs.

c) Contributions to Global Landslide Issues
The IPL will mobilize global cooperation for strengthening research and learning on risk reduction for landslides and related earth system disasters at sites identified as of great concern to the global community, such as Macchu-Picchu, the Kashmir, Central Asia high mountainous area, and Bamiyan.
d) **Partnerships**

Mutually beneficial partnerships with other global initiatives, such as the International Hydrological Program (IHP), the International Geoscience Program (IGCP), and the Mountain Partnership will be developed.
LANDSLIDE RISK MANAGEMENT IN THE UNITED KINGDOM

M. G. Winter
Transport Research Laboratory, United Kingdom

R. G. McInnes
Coastal and Geotechnical Services, United Kingdom

E. N. Bromhead
Kingston University, United Kingdom

Abstract: The United Kingdom (UK) is a geologically and geomorphologically diverse but small country and a wide range of processes, including landslides, may be observed within its boundaries. In the south of England, the Mesozoic and Tertiary clays commonly give rise to landsliding as typified by the deep-seated, slow-moving/episodic, reactivated features associated with the Cretaceous deposits of the coastal landslide complex of the Isle of Wight which pose substantial risk mainly to property and local infrastructure. In the northern part of the UK, in particular in Scotland, shallow, rapid debris flows associated with the more recent Upper Pleistocene glacial deposits pose a risk to both life and limb, and to the strategic road network serving often remote, but sizable, communities. The bulk of these debris flows are not reactivated but form part of the process of hillside wasting. This paper compares and contrasts the hazards and risks associated with the landslides of the Isle of Wight Undercliff and those associated with Scottish debris flows as they interact with the road network. The approaches to landslide risk management in these contrasting environments are very different. Social, economic and environmental considerations are discussed alongside the potential effects of climate change on the outlook for landslide activity in the UK and the associated need for robust risk management strategies.

INTRODUCTION

In the United Kingdom (UK), the infrastructure (road, rail, canal, etc) operators have each evolved strategies for asset management associated with structures, earthworks and other elements. In the case of earthworks, these are typically as reviewed by CIRIA (Perry et al. 2003a; 2003b). However, these asset management strategies are rather concentrated on man-made earthworks (cuttings and embankments). A typical example of this is the DMRB (HD41/03) for the maintenance of geotechnical assets (Anon 2003). It seems likely that climate change will affect natural slopes to a greater degree than man-made earthworks and it is therefore timely to draw attention to the issues associated with asset management in a changing environment dominated by natural slopes.

The Isle of Wight is located off the south coast of England (Figure 1). ‘The Undercliff’ is an ancient landslide complex which stretches along some 12km of the south coast of the Isle of Wight, ranging inland by between 500 to 1,000m; it forms the largest urban landslide complex in north-western Europe. The landslide complex developed in Lower and Upper Cretaceous rocks during two main phases of landsliding which are thought to have taken place, around 8,000 to 4,500 years ago and 2,500 to 1,800 years ago, after the last Ice Age. These phases of landslide activity followed significant changes in climate and sea level rise and the resulting impacts of coastal erosion along the Isle of Wight’s southern coastline.
(Hutchinson & Bromhead 2002). The Isle of Wight Council’s Undercliff Landslide Management Strategy aims to reduce the likelihood of future ground movements by seeking to control the causative factors and by limiting the impact of future movement through the adoption of good practice and appropriate planning and building controls. A landslide quantitative risk assessment was completed in 2006 (Halcrow 2006).

Figure 1: Map showing the location of the Isle of Wight and Scotland within the UK

Debris flows in Scotland (Figure 1) form part of the process of hillside wasting that has been ongoing since the most recent ice retreat. They generally occur in relatively remote areas and the major risks are associated with the transport infrastructure and its users, particularly the strategic road network. The strategic road network, while passing often through remote areas, links Scotland’s cities and major towns and underpins much of Scotland’s socio-economic activity including tourism. To date, risk management efforts have focussed primarily upon this network and the activities associated with it. A GIS-based evaluation has been conducted
to determine the most hazardous areas in respect of debris flow formation and the data interpreted to indicate the associated hazard to the strategic road network (Winter et al. in press). The amount of traffic and the availability/difficulty of traffic diversion are used to determine the exposure (the vulnerability of the elements at risk) and thus the risk. Management is within the framework of detection-notification-action (DNA), with the emphasis on exposure reduction at times of actual and heightened likelihood of debris flow activity.

In this paper the major hazards and risks associated with each of these landslide types are described and the different approaches to landslide risk management are described, compared and contrasted. Robust risk management strategies have been developed in the context of social, economic and environmental considerations and the potential effects of climate change on the outlook for landslide activity in the UK.

LANDSLIDE HAZARDS

Undercliff, Isle of Wight
The striking scenery of the Isle of Wight Undercliff, which is so important to the Island’s visitor-based economy, is strongly influenced by the underlying geology, landsliding and coastal erosion processes. The Isle of Wight Council has, over many years, been developing and implementing strategies for the sustainable management of coastal instability risk. This has involved leading-edge research into the fundamental causes of the instability (Hutchinson & Bromhead 2002) and techniques for the management of large landslides in urban areas (McInnes & Jakeways 2000a). The environmentally-friendly coastal and geotechnical engineering schemes help manage the risk while maintaining the Island’s coastal environment.

The landslides of the Isle of Wight Undercliff are deep-seated and generally slow moving, which has allowed the historical development of the area, particularly since the mid-19th century. However, the impact of landslide movement and its long term effect on the urban environment is a serious problem. There is ongoing damage to the infrastructure including roads, retaining walls, underground services and buildings. In fact, over the last 100 years, some 50 properties have had to be demolished due to the impacts of ground movement whilst others have sustained significant damage. The annual cost of landslide damage and management measures is estimated to exceed £3 million a year. Despite this, some locations within the Undercliff have remained relatively stable with ancient structures including churches dating from the 11th century and earlier, as well as stone farmhouses and substantial Victorian buildings, which have remained comparatively unaffected. As an example, the St. Catherine’s Point lighthouse has been shown to be moving on a sub-horizontal basal slip surface. Thus, despite considerable seaward movement over nearly two centuries it has remained essentially vertical and thus able to function as a working lighthouse (Hutchinson et al. 2002). Although in the Winter of 2000-2001, the lighthouse experienced seaward movement of about 100mm, much more serious in terms of impact on the island’s economy was the series of mudslides that damaged and in places severed the A3055 Undercliff Drive road (Figure 2).

Scottish Road Network
Scotland contains some of the Europe’s most attractive and ecologically important mountain environments, including two National Parks. In August 2004, a series of rainfall-induced
debris flows affected important parts of the major road network that links both cities and smaller, remote communities. While debris flows occur with some frequency in Scotland, they have, in the past, relatively rarely affected major infrastructure. However, when they do impact upon the road network, for example, the degree of damage and the loss of utility to road users, can have a major detrimental effect on the economic and social aspects of the use of the asset. There is also the potential for such events to cause serious injury and even loss of life, although, fortuitously, such consequences have been limited to date; for example, the August 2004 events caused no fatalities or major injuries (Winter et al. 2005a).

The impacts of such events are particularly serious during the summer months due to the major contribution that tourism makes to Scotland's economy. Nevertheless, the impacts of debris flow events during the winter months should not be underestimated. Not surprisingly, these debris flow events created a high profile for the effects of landslide activity in the media.

The rainfall in August 2004 was substantially in excess of the norm; some areas received over 300% of the 30-year monthly average (source: www.metoffice.gov.uk). Subsequent analysis of radar data indicated that at Callander, some 20km distant from the events at the A85, some 85mm of rain fell during a four-hour period on 18 August with a peak intensity of 150mm/hour. The 30-year average rainfall for August in Scotland varies between 67mm on the east coast and 150mm in the west of Scotland (Anon 1989).

The rainfall was both intense and long lasting and many debris flows were experienced. A small number of these intersected the major road network, notably the A83 between Glen Kinglas and to the north of Cairndow on 9 August, the A9 to the north of Dunkeld on 11 August, and the A85 at Glen Ogle on 18 August (Figure 3).

While there were no major injuries, some 57 people were taken to safety by helicopter after
being trapped on the A85 in Glen Ogle (Figure 4). The events of August 2004 are described in detail by Winter et al. (2006a; 2006b).

Not only are such events not uncommon in Scotland, they have occurred over a long period of time. Radiocarbon dating of materials associated with debris flow activity by Innes (1982) indicates that such activity has occurred throughout most of the last 7,000 years, albeit with clustering of the data which may be indicative of periods of higher and lower levels of activity (Ballantyne 2004). However, based upon stratigraphic evidence, Ballantyne (2004) also suggests that such activity was widespread after ice-sheet deglaciation and during and after ice retreat at the end of the Loch Lomond Stade approximately 11,500 years ago. What is perhaps unusual, in the recent past at least, is the impact that the events have had upon the strategic transport network in Scotland. In the recent past, such impacts have largely been confined to the local road network, some of which are described along with the responses to the events by Nettleton et al. (2005).

Figure 3: Map showing Scotland’s main roads and locations of debris flows in August 2004
LANDSLIDE RISK MANAGEMENT

Undercliff, Isle of Wight
In the late 1980s, the Department of the Environment (DOE) commissioned a pilot study of central Ventnor (DOE 1991) to assist the development of planning policy guidance for ‘development on unstable land’. Subsequent studies were undertaken in order to:

- Determine the nature and extent of the landsliding problems.
- Understand the past behaviour of separate parts of the Undercliff.
- Formulate a range of management strategies to try and reduce the impact of ground movement.

The programme of work comprised a thorough review of available records, reports and documents followed by a programme of detailed field investigations involving geomorphological and geological mapping, assessments of ground movement rates, a survey of damage caused by ground movement, and a review of local building practices; this has been followed more recently by detailed sub-surface investigations.

Figure 4: Oblique aerial view of the northerly debris flow at A85 Glen Ogle

The Isle of Wight Council and its predecessors subsequently commissioned several extensions of the original DOE study of central Ventnor to provide coverage for the whole of the Undercliff (see Figure 5 and 6). The recommendations arising from the studies were developed into a ‘Landslide Management Strategy’ which has assisted the Council in seeking to manage the instability problems, through, in particular, improved drainage, additional coast protection works, landslide monitoring and prediction, and further research (see Figure 7).
Figure 5: Phased studies for the Isle of Wight Undercliff

Figure 6: Aerial view of part of the Isle of Wight Undercliff landslide complex
(Courtesy of the Isle of Wight Centre for the Coastal Environment/Wight Light Gallery)
A particular feature of previous studies has been the dissemination of the findings for both the general public, to increase their knowledge and awareness of the instability problems, and to professional practitioners in landslide management through a series of publications, exhibitions at the Isle of Wight Coastal Visitors’ Centre and presentations at international conferences. In fact, three conferences have been held on the Isle of Wight (in 1991, 2002, and in 2007 a third four day conference entitled ‘Landslides and climate change – Challenges and solutions’) also took place in Ventnor.

The landslide management strategy and previous work have recognised the need for detailed sub-surface investigations to verify the mechanisms and causes of coastal landsliding. Initially in 2002, and continuing in 2005 with financial support from the Department for Transport, the Isle of Wight Council commissioned a programme of ground investigations in central Ventnor in order to prepare an interpretative report from the findings including a Quantitative Risk Assessment (QRA). This report (Halcrow 2006) has advanced the understanding of the mechanisms and causes of coastal landsliding in central Ventnor and, for the first time, the QRA evaluates the likelihood and consequences of future risk scenarios. This work represents the first phase and conceptual design for a possible civil engineering solution following an option analysis by a team comprising many of the UK’s experts in this field.

In those areas that have suffered damage, it is also the case that the foundations and building styles are often unsuited to accommodating ground movement. However, it is important to recognise the predicted impacts of climate change which may result in an increase of winter rainfall of between 26 to 30% by the year 2080. The consequences of this are being investigated by the Council in order to seek a solution to mitigate such adverse effects.

A comprehensive examination of coastal landslide potential within part of the Ventnor Undercliff was commissioned by the former Department of the Environment in 1987 (DOE 1991). This three year study commenced with a review of available records, reports and documents followed by the preparation of geomorphological maps based on field surveying. Furthermore, a survey of damage caused by ground movement including a visual inspection

---

**Figure 7: Isle of Wight landslide management strategy**

A particular feature of previous studies has been the dissemination of the findings for both the general public, to increase their knowledge and awareness of the instability problems, and to professional practitioners in landslide management through a series of publications, exhibitions at the Isle of Wight Coastal Visitors’ Centre and presentations at international conferences. In fact, three conferences have been held on the Isle of Wight (in 1991, 2002, and in 2007 a third four day conference entitled ‘Landslides and climate change – Challenges and solutions’) also took place in Ventnor.

The landslide management strategy and previous work have recognised the need for detailed sub-surface investigations to verify the mechanisms and causes of coastal landsliding. Initially in 2002, and continuing in 2005 with financial support from the Department for Transport, the Isle of Wight Council commissioned a programme of ground investigations in central Ventnor in order to prepare an interpretative report from the findings including a Quantitative Risk Assessment (QRA). This report (Halcrow 2006) has advanced the understanding of the mechanisms and causes of coastal landsliding in central Ventnor and, for the first time, the QRA evaluates the likelihood and consequences of future risk scenarios. This work represents the first phase and conceptual design for a possible civil engineering solution following an option analysis by a team comprising many of the UK’s experts in this field.

In those areas that have suffered damage, it is also the case that the foundations and building styles are often unsuited to accommodating ground movement. However, it is important to recognise the predicted impacts of climate change which may result in an increase of winter rainfall of between 26 to 30% by the year 2080. The consequences of this are being investigated by the Council in order to seek a solution to mitigate such adverse effects.

A comprehensive examination of coastal landslide potential within part of the Ventnor Undercliff was commissioned by the former Department of the Environment in 1987 (DOE 1991). This three year study commenced with a review of available records, reports and documents followed by the preparation of geomorphological maps based on field surveying. Furthermore, a survey of damage caused by ground movement including a visual inspection
of damage to roads, walls, buildings and information on underground services, assisted in understanding the extent of the problem. These surveys were supported by a land-use survey and a review of local building practices. This information, also drawing upon the results of past site investigations, assisted in identifying the nature and extent of landsliding within the Undercliff and the types of contemporary movements taking place including the magnitude and frequency of events and their impact. Furthermore, the information assisted in assessing the nature of land use at risk and the vulnerability of structures to ground movements of different intensities.

All the information was incorporated in a GIS which allowed the factors influencing the distribution and frequency of contemporary movements to be summarised on a ‘ground behaviour map’.

As part of this review, it was necessary to identify which key factors were responsible for promoting landslide activity in the Undercliff, including consideration of the natural and human processes that might accelerate ground movement, and to highlight the changes that have resulted from these activities. Clearly the impact of development and human activity within the Undercliff has been a significant contributory factor in terms of ground movement events.

The nature of the landslide hazard faced by the Undercliff community has been defined by producing maps of contemporary ground behaviour. These maps have been prepared drawing on the information such as records of damage caused by ground movement as well as data from other sources. Through these methods, an understanding of the following components of landslide hazard and risk has been achieved:

- The extent of the landslide complex, systems and the processes involved in its evolution.
- The types of contemporary ground movement taking place.
- The magnitude of contemporary ground movements.
- The frequency of landslide events.
- The causes of landslide events and their temporal variation.
- The impacts of ground movement in built-up areas.
- The nature and extent of property at risk.
- The vulnerability of different styles of construction to ground movement.

Recent ground investigations involving the drilling of a series of boreholes in a line from Upper Ventnor extending down through the town to the seafront has confirmed much of the surface geomorphological assessment work undertaken between 1988 and 1991 as well as providing valuable additional information.

Following the completion of the Department for the Environment study, the Isle of Wight Council’s predecessor commissioned further studies allowing the whole of the Undercliff to be mapped to a similar standard.

The mapping process involved the preparation of three principal map types:

- Geomorphological map provides a summary of the surface form of the landslide and the surrounding area and shows the relative positions of the main geomorphological units occurring within the landslide, as well as the nature and extent of individual landslide units. This information provides a foundation for study and investigation of landslides such as
the Undercliff.

- Ground behaviour map seeks to define the hazard resulting from ground movement.
- Planning guidance maps were developed to provide information to support the statutory planning system.

It is recognised that some areas are unsuitable for development and a policy of avoidance has to be implemented. With this additional information, informed decisions can be made through the planning process, which will guide development away from areas at risk and assist in avoiding problems arising from ground instability in the future.

Since 1993 the Isle of Wight Council has encouraged a co-ordinated approach to the management of ground instability problems through its ‘Landslip Management Committee’. This brings together stakeholder interests including local authority coastal management, planning, building control and highways staff, estate agents, construction industry representatives, service companies, insurers and residents. Ground instability in the Undercliff has, in fact, been addressed in a number of ways but the key tasks have been:

- Preventing unsuitable development through sound planning controls and building control measures.
- Monitoring ground movements and weather conditions using a range of automatic and manual recording instruments and stations.
- Seeking to improve ground conditions through a range of measures aimed at controlling water in the ground as well as coast protection schemes which reduce marine erosion at the toe of the landslide.
- A major awareness-raising programme for the benefit of both professionals and the general public living and working in the area.

In recent years, scientific research has provided a wealth of additional technology and techniques that have assisted the understanding of ground instability problems (Leroi et al. 2005). At a local level, lessons learnt from a number of major landslide events on the Isle of Wight have also highlighted the role that human activity can play in instigating ground movements. As development pressures have increased the occupation of more stable and commercially attractive locations, there has been a demand also to extend new development into adjacent areas where there may be greater physical constraints because of problems such as instability. Such developments, particularly in the absence of effective planning policies, may lead to costly mistakes posing risks to life and property. These can usually be avoided if appropriate planning and landslide management strategies are in place, which have taken full account of ground conditions in their formulation.

It has been concluded (Jones & Lee 1994) that landslide problems are not ‘acts of god’ or unpredictable, entirely natural events that can at best only be resolved by avoidance or large-scale civil engineering works. The role of human activity in initiating or reactivating many coastal slope problems should not be under-estimated. In areas such as the Isle of Wight where urban development has taken place on coastal landslides, the problems tend to be related to slow ground movement and progressive damage to property, services and infrastructure (Doornkamp et al. 1991). In such circumstances, many problems can be reduced if there is a programme of active landslide management in place, where the local community is able to come to terms with the situation and learn to ‘live with landslides’ (Lee 1997; McInnes 2000).
Scottish Road Network
Following the events of August 2004 and, indeed, events prior to that date, the need to act was recognised by Transport Scotland in order to ensure that in the future there is a system in place for assessing the hazards posed by debris flows. The system must be capable of ranking the hazards in terms of their potential relative effects on road users. Thus enabling the future effects of debris flow events to be managed and mitigated as appropriate and as budgets permit, ensuring that the exposure of road users to the consequences of future debris flows is minimized whilst acknowledging that it is not possible to prevent the occurrence of such events.

As a first, but significant, step towards that overall objective, an initial study was set in motion by the then Minister for Transport, Nicol Stephen MSP, in recognition of the national importance of the problem to address the following activities:

- Considering the options for undertaking a detailed review of side slopes adjacent to the trunk road network and recommending a course of action.
- Outlining possible mitigation measures and management strategies that might be adopted.
- Undertaking an initial review to identify obvious areas that have the greatest potential for similar events in the future.

Key tasks were undertaken by members of a Working Group and the results were incorporated into a technical report (Winter et al. 2005a) and a summary report (Winter et al. 2005b) designed to inform a wider audience of the actions both since the events of August 2004 and planned for the future.

This work led to the second part of the study, which included the development of a system to allow a detailed review of the network and the identification of locations of greatest hazard; for those hazards to be ranked and appropriate mitigation and/or management measures to be selected. Initially, the methodology for the assessment of hazard and exposure was developed to provide a hazard ranking, together with a range of appropriate management options (Winter et al. 2007a). Figure 8 presents a flow chart of the work, which forms three main elements, summarised as follows:

- Undertaking a GIS-based assessment as a first stage in the hazard assessment process.
- Undertaking site specific hazard and exposure assessments of areas identified by the GIS as being of higher hazard.
- The identification and development of appropriate management processes for each category of hazard ranking.
The GIS-based assessment enabled site specific assessments to be targeted in order to obtain better value from resource-intensive activities. It also allowed the elimination of large areas having minimal hazard. Figure 9 illustrates the results for Glen Ogle. This method of assessment has proved to be a valuable tool for assessing debris flow hazards, but does require considerable interpretation in order to relate those hazards to the infrastructure.

This assessment method has allowed specific lengths of routes to be identified and prioritized for site-specific consideration, comprising (1) a familiarization process including examination of available topographic survey information and the GIS-based assessment, (2) aerial photography, and (3) a brief site visit. In some cases, this has proved sufficient to determine any adjustment required to the hazard assessment, while in others, a more detailed site walkover has been required. The first tranche of site-based assessments were conducted during the summer of 2007, with the basic principle being applied that hazard scores were adjusted on the basis of information that was either different from, or was not available to, the GIS-based assessment and its interpretation.

Exposure of road users to such hazards is assessed by means of the traffic flow and the potential for disruption to traffic that would occur if a diversion must be implemented. The exposure of road users is taken as an analogue for the vulnerability of the elements at risk – the elements at risk being a simple ‘binary switch’ as a road user is either present or not. This simple approach to exposure encompasses both life-and-limb vulnerability and also a large element of socio-economic vulnerability from the disruption to traffic inherent in both the diversionary and the traffic level scores. It is, however, clear that not all socio-economic vulnerabilities are captured and in order to recognize this, and also to provide compatibility with existing assessment measures in use on the Scottish trunk road network (e.g. McMillan & Matheson 1997), the term ‘hazard ranking’ is used in place of risk. Once a hazard ranking
has been derived, an appropriate management option may be selected from the range of options under development. In essence, four levels of action are envisaged and are, indeed, being implemented in selected areas.

Figure 9: A sample of the GIS-based hazard assessment: Glen Ogle (tick marks 1km apart)

The ‘Do-Nothing’ approach is intended to be applied to sites of low hazard ranking for which substantial expenditure is inappropriate. The ‘Do-Minimum’ option, with the potential to mitigate the impacts of debris flows to some extent, involves simply ensuring that forward plans are in place to ensure that diversion routes are available and may be exploited in an expeditious and well organised manner. Diversion route maps and contingency plans are currently held for the main road network. In the case of both the ‘Do-Nothing’ and ‘Do-Minimum’ options, it is not possible to eliminate the chance of a debris flow event affecting such areas but any occurrence is seen as unlikely and largely unforeseeable. Any residual exposure cannot readily be quantified and is unlikely to justify the commitment of additional resources which may be better expended at other locations.

‘Do-Something 1’ is the first management option where site specific action is contemplated. Such action is essentially exposure reduction by managing the access to and/or actions of the road-using public on the network at times either when events occur or precursor rainfall has indicated a high likelihood of debris flows occurring. ‘Do-Something 2’ involves physical works in order to achieve hazard reduction. The approaches involved include physical measures, such as the protection of the road, reduction of the opportunity for a debris flow to
occur or realignment of the road away from the area of high hazard. Such options need to be considered in the context of the policy governing Transport Scotland’s overall main road maintenance and construction programme. Exposure reduction and hazard reduction are specifically addressed below.

Clearly, and as illustrated in Figure 8, ‘Monitoring and Feedback’ is fundamental to the success of the system and key to deriving best value from the arrangements proposed. The system is an active one and lessons learned from future debris flow events, whether they occur in areas of high or very high hazard ranking or not, will produce valuable data which needs to be taken into account in adjusting the parameters that form the cornerstone of the assessment methodology.

Processes described above culminate in a decision on whether the hazard ranking, in the context of the safe operation of the road network at any location, is acceptable or not. At those locations where the hazard ranking is unacceptable, some form of mitigative action is required. To reduce the hazard ranking (or risk) to the road user to acceptable levels, either the magnitude of the hazard and/or the potential exposure, or losses that are likely to arise as a result of debris flow, must be reduced.

The reduction of the exposure of road users forms the main focus here of the work here. In this case, the debris flow event is taken as a given hazard and either the number of people exposed to the hazard must be reduced, for example, by closure of the road, or they must be warned to exercise caution at appropriate times and places. To reduce the hazard, physical intervention is required. In many cases, there will be higher cost and more intrusive options and it is anticipated that relatively few locations will justify such expenditure.

Intended reduction of exposure as a policy lends itself to the use of a simple and memorable three-part management tool (Winter et al. 2005a) as Detection-Notification-Action (DNA), as follows:

- **Detection**: The identification of either the occurrence of an event by instrumentation/monitoring, observation, or by the measurement and/or forecast of precursor conditions (e.g. rainfall).
- **Notification**: The notification of either the likely or actual occurrence of an incident to the authorities, including the Police, Traffic Scotland, Transport Scotland and the relevant Operating Company.
- **Action**: The proactive process by which intervention reduces the exposure of the road user to the hazard, for example by road closure or traffic diversion.

In the short-term to medium-term, the DNA approach to mitigation must be reactive to debris flow events. There may be a case for reacting to extremely heavy rainfall events. However, a caveat to this is the need to consider carefully at what levels the triggers should be set, in so far as the relationship between rainfall and landslides/debris flows in Scotland is by no means fully understood. In the longer-term, the detection of precursor triggering conditions (i.e. rainfall) may enable both the notification and action phases to be taken prior to the occurrence of major events. Such an approach is discussed in more detail by Winter et al. (2007b).

Detection: The movement of slope material can be monitored in real time and used as a management tool. Monitoring instruments, such as tilt meters and acoustic sensors, can be
located so as to record movement from potential debris flow or positioned such that notification is received if debris reaches or gets close to a road (e.g. trip wires).

In relation to the former, the seeding area for debris flows can be very large and high on the hillside. This introduces considerable difficulty in pinpointing the optimum location for the installation of the monitoring system and doubt as to whether the debris will reach the road. An instrumented fence is used on the Scottish rail network at Glen Douglas to recognise when material impinges upon the line. Similarly, a system to detect rock falls and debris flows is used at the Pass of Brander above the A85 at Loch Awe to shut the line and stop trains. Whether such a system is sufficient in isolation is questionable but it is considered that in conjunction with rainfall monitoring and possibly the deployment of operatives, the likelihood of road users being affected by debris flow events could be reduced significantly. The range of possible electronic instrumentation types (data sent to a central control point) is presented in Winter et al. (2005a) but includes borehole or shallow inclinometers, tilt meters, ‘trip wire’, ‘ball of string’, acoustic meters, and remote sensing.

An alternative approach is to use visual observation to detect debris flow events either by closed-circuit television or, more practically for long stretches of hillside, by introducing “landslide patrols” during periods of high rainfall. It is essential that such operatives are trained in what to look for and that patrols should operate in pairs for safety reasons. Given the wide range of locations at which debris flow activity may be experienced this might prove to be a more practical alternative, the costs of instrumenting and monitoring extensive lengths of slope being potentially prohibitive. In addition, the issue of inadvertent activation of systems, such as trip wires by, for example, hill-walkers, would need to be addressed in the context of the road network, to which access is less constrained than that to the rail network. The value of observations made by the general public should also not be underestimated, especially given the proliferation and ubiquity of mobile telecommunications.

Notification: In the immediate aftermath of the occurrence of a debris flow event, notification must be sent to the Police, the Operating Company and the infrastructure owner. The decision must then be made rapidly as to what action is to be taken (see below). The nature of debris flows is such that in most cases the road will be blocked and therefore closed to all intents and purposes.

It is important to note that, if “landslide patrols” are used as part of the ‘Detection’ process, then that role must also be extended to ensure that the proper authorities are notified promptly.

Action: Following a debris flow, a number of positive options for action are available. Firstly, the road length (or lengths) affected could be closed and appropriate pre-planned diversion routes put in place. However, it is important to note that closing the road in the area immediately adjacent to the event is not an adequate response. Debris flow propensity generally affects long lengths of hillside and an evaluation of the vulnerable area must be performed in order to ensure that an appropriate length of road is closed.

Closure might be achieved by installing barriers such as the snow barriers present on some of Scotland’s roads (Figure 10). Such an approach is applied to the Sea-to-Sky Highway in British Columbia, a route well-known for its propensity to disruption due to debris flow, and gates are in place on this route for the specific purpose of closing the road in the event of debris flow (see Winter et al. 2006b). A road closure may only be ordered by the Police.
However, in practice, such decisions are made in consultation with and/or on the recommendation of other appropriate bodies. In this case, such bodies are likely to include Traffic Scotland, who collate and distribute information about the factors likely to affect traffic flows, and the relevant Operating Company.

Figure 10: Snow gates on the A9. These are used to close the road over Drumochter Pass when it is impassable due to snowfall. These and similar installations could also be used to close the road in the event of debris flow.

Warning the public of hazards is an important feature of any ‘Action’ programme and landslide patrols can only heighten the awareness of road users of potential hazards. In Scotland, there is a variety of potential means of making public announcements when either debris flows have occurred or there is heightened likelihood of their occurrence in an area. This might involve systems such as traffic information websites, variable message signs (VMS) (Figure 11) and media (radio, TV and web) announcements, notifying drivers that their potential exposure to the hazards posed by debris flows is real and present. Announcements could also be linked to traffic guidance systems. Static signs may also be used to convey both general information on the nature and locations of potential hazards and also to convey specific instructions (Winter et al. 2007a).

It is important to ensure that measures such as the provision of signs, especially VMS, and landslide gates must be at strategic locations on the network. Such locations should allow drivers to make a viable decision as to whether they proceed with their planned route, use an alternative or return to their point of departure. For example, the snow gates illustrated in Figure 10 are located at the beginning of a stretch of dual carriageway where provision is made for traffic to turn around and the VMS sign illustrated in Figure 11 is located on the approach to a major interchange.
Figure 11: A VMS located on the A9. The Scottish VMS network is being significantly increased, forming a crucial part of the hazard warning strategy for road users

In all cases, re-opening of the road, or its return to normal operation, can only occur after a thorough inspection of the road and the adjacent slopes has been undertaken to ensure that the likelihood of further debris flow events is at an acceptable level. Current practice is to undertake ground-based inspections only when the adverse weather has abated and only to re-open the road once such inspections indicate that the residual hazard and exposure are at an acceptable level.

A predictive capability is under development in Scotland (Winter et al. 2007b) in order to enable periods of heightened risk due to high rainfall to be identified. This will allow a more rapid response, potentially in advance of events, and thus enable reductions in the exposure of road users to the effects of debris flows.

Hazard reduction presents a challenge in identifying locations that are of sufficiently high hazard ranking to warrant spending significant sums of money on engineering works. The costs associated with installing remedial works over long lengths of road are difficult to justify. Moreover the environmental impact of such engineering work should not be underestimated, having at the least a lasting visual impact.

Typically, the reduction in hazard will entail physical engineering works to change the nature of a slope or road to reduce the potential for either initiation and/or the potential for a debris flow to reach the road. Hence, there are three broad approaches to selection of hazard reduction works:

- Accept that debris flows will occur and protect the road. Potential solutions include debris basins, lined debris channels, debris flow shelters, overshoots and barrier fences.
- Carry out engineering works to reduce the opportunity for a debris flow to occur.
- Realign the road.

These options are reviewed in more detail by Winter et al. (2005a; 2006b; 2007a) in the context of this project and the Scottish environment. It is anticipated that few in any such actions will be appropriate to the widely-dispersed hazards extant on the Scottish trunk road network and that their use should be limited to locations where their worth can be demonstrated within the broader context of construction and maintenance budgets and priorities.

However, simple measures such as ensuring that channels and gullies are kept open can be effective in terms of hazard reduction. This requires that the maintenance regime is fully effective both in routine terms and also in response to periods of high rainfall, flood and slope movement. It is also important that maintenance and construction projects currently in design take the opportunity to limit any hazards by incorporating, where suitable, measures such as higher capacity or better forms of drainage, or debris traps. In particular, critical review of the alignment of culverts and other conduits close to the road should be carried out as part of any planned maintenance or construction activities.

**COMPARISON OF APPROACHES TO LANDSLIDE RISK MANAGEMENT**

The affirmation that we live in a ‘risk averse society’ is becoming common and implies that the willingness to accept, or to tolerate, risk is low. In many spheres of life, such a statement may well be accurate, however it remains relatively meaningless unless it is viewed in the broader context. Such a context includes the willingness (and/or ability) of society to pay for risk reduction measures and the willingness to alter the environment in order to accommodate such measures.

Provided that the willingness to accept risk, pay and alter the environment can be described at a conceptual level, the approaches in different parts of the world and in different situations may be simply and graphically compared to gain a deeper understanding of the drivers for the approach to risk mitigation. This is achieved by means of the ternary ‘Willingness Diagram’ (Winter et al. in press) (Figure 12). The ‘Willingness Diagram’ inter-relates three parameters, thus constraining any one of the three in terms of the levels assigned to the other two. Thus the assumption is implicit that there is a fixed amount of ‘willingness’ to share between the following parameters:

1. Willingness to accept (or tolerate) risk.
2. Willingness (and/or ability) to pay.
3. Willingness to alter the environment in the pursuit of lower risk.

It is important to note that the diagram is not intended to highlight either incorrect or correct approaches. It is intended to reflect different approaches which may be the result of a wide range of inputs to the decision-making process, including engineering, geological, geomorphological, economic, data, information, sociological and cultural factors.

In the Isle of Wight, the willingness to accept risk is relatively low and the willingness to pay relatively high, despite the fact that the risks are generally to property rather than to life and limb. At the same time, the willingness to affect the environment is relatively low and these factors drive the use of the generally discrete and ‘invisible’ solutions that are implemented.
Since the year 1900, over 50 properties have been lost, with damage to many others and to the local infrastructure, as a result of slow landslide ground movement in the Isle of Wight; these and other impacts amount to an estimated annual cost to the economy of £3 million per annum. The impacts of uncertainty over ground conditions and the resulting knock-on effects on the availability of property insurance and the local economy generally were key drivers for research into the nature and scale of the ground movement problems. This coincided with the desire by the government to develop planning policy guidance for areas affected by instability and the commission of the ‘Ventnor Study’ (DOE 1991). This initiative was then taken up by the Isle of Wight Council, which extended the study area and developed a ‘Landslide Management Strategy’.

In respect of Scotland’s roads, the both the willingness to affect the environment and the willingness to pay are relatively low, and management solutions are thus favoured over intrusive engineering solutions. There is thus an acceptance of a certain level of risk, although these risks are generally significantly less than those posed by road traffic accidents, for example.

In Scotland, some of the key drivers for the willingness to accept risk are social, economic and environmental, often including elements of all three. Roads in Scotland provide vital communication links to remote communities from both the social and economic viewpoint and the severance of the communities from services and markets for goods is highly

Figure 12: The ‘Willingness Diagram’ showing the different approaches to landslide risk in the UK and other parts of the World
undesirable. An example of the adverse impacts that severance may have on communities may be drawn from Jamaica. In this case (Figure 13), a landslide has occurred on the B1 route in the Blue Mountains in Jamaica effectively severing the local coffee production industry from the most direct route to markets accessed from the island’s north coast.

The landscape has both a social and an environmental value, but what is often forgotten is that its economic value to Scotland is substantial as it attracts much business in the form of tourism and is especially important to many of the remote communities potentially affected by landslides. The height of the tourist season coincides with the summer landslide season of July and August and thus, in parallel with the need to maintain access, detrimental effects on tourism from negative publicity are unwelcome to both politicians and the public. At the same time, adverse visual impacts on the landscape by large defence/remediation structures (e.g. debris basins, overshoots, shelters, etc.) are seen as undesirable. Thus, the underlying philosophy of any remediation must be to preserve the natural landscape as much as is possible even if only to protect when tourist come to visit.

Figure 13: Landslide on the B1 road at section in Portland Parish, Jamaica

The avoidance of adverse impacts on other valuable natural resources is also a key issue. Examples of such impacts might include the alteration of the hydrogeological regime of protected peat bogs and adding silt to protected/valuable salmon fishing/spawning rivers.
It is useful to contrast the UK approaches outlined above with some other international examples.

The United States of America is often cited as a good, if not definitive, example of a risk averse society. However, the evidence does not always support this assertion. For example, the DeBeque Canyon landslide affects Interstate 70, the main east-west route through Colorado (Figure 14). During the last reactivation of the landslide in April 1998, the road heaved 4.3m and shifted 3m laterally towards the nearby river (White et al. 2007). The landslide continues to move possibly forewarning of future rockslides from above and heaving rotation failures of the road. The Colorado Department of Transportation (CODoT) have undertaken a series of remediation measures as described by White et al. (2007) and commissioned a long-term monitoring system. However, the overall approach seems to be that the movements described above are at an acceptable level and can be managed on an emergency works basis as and when they happen.

The example of DeBeque Canyon, cited above, implies a high level of willingness to accept risk and an associated low level of willingness to pay, possibly driven by an unwillingness to affect the environment and, potentially, higher levels of risk elsewhere which may take priority.

The situation in Hong Kong, where life has been valued at a high, but nonetheless realistic, level and the willingness to accept risk is relatively low, provides an interesting counterpoint. In the 1980s, the willingness to affect the environment was also at a relatively high level with hard engineering solutions often dominating the scene (e.g. Figure 15), and this most likely tempered the costs and therefore the willingness to pay. In the latter part of the 1990s and beyond, there was an apparent shift in the approach in Hong Kong and the willingness to affect the environment was much reduced leading to softer vegetative solutions where appropriate. This may have been associated with an increase in the willingness to accept risk as some of the solutions used may be less robust and with a potential increase in cost, and thus willingness to pay, if only in terms of an increase in the maintenance expenditure required for such soft solutions.
In Japan, the willingness to accept risk is low. Take for example the case of the Zentoku landslide on Shikoku Island, stabilised with a combination of 20 nos. of 5.45m diameter concrete piles ranging in depth up to 44m, 280 ground anchors and six drainage shafts with bored drainage arrays (Hong et al. 2005; Bromhead 1997); clearly cost was not the significant factor. The willingness to significantly modify the environment with intrusive engineering works and the unwillingness to accept even relatively low levels of risk combined to make the high costs involved almost a side issue – in some respects this extreme example almost renders the ‘Willingness Diagram’ redundant.

In general terms, bioengineering represents a least cost and possibly high risk solution to many slope instability problems, engineering earthworks often represent comparatively low cost but are rather intrusive and often retain a residual risk and tunnelling for drainage works is a high cost solution but one which least affects the ground surface environment while reducing the risk associated with future ground movements. This selection is placed in indicative positions on the ‘Willingness Diagram’ and shows the positioning of the range of approaches practised in the UK and elsewhere.

![Figure 15: A shotcrete slope in Kowloon, Hong Kong SAR](image-url)

**THE PUBLIC: THE ULTIMATE STAKEHOLDER**

To implement the Landslide Management Strategy, a Management Committee, comprising key professionals from the Council (coastal management, highways, planning and building control), the water industry and other service industries, surveyors, estate agents and insurers, meet twice a year. The Committee assesses progress on implementation of the Strategy and
exchanges information on new initiatives being led by the Isle of Wight Council and others.

There has been a long history of consultations over ground conditions within the Ventnor Undercliff and, therefore, many residents are aware of the geological situation. The town is extremely attractively located with development on the various landslide benches offering panoramic sea views over the Victorian town, the adjacent spectacular coastline and the English Channel and the property market is healthy.

On completion of the DOE study in 1991 the first of a range of display boards were assembled covering the following themes:

- What is the history of ground movement in the area?
- What is the scale of the problem?
- Why is there a ground movement problem?
- What causes ground movement?
- How can we define landslide hazard?
- How can the landslide problems be managed most effectively?
- What is the local authority doing to help?
- What can developers do?
- What can estate agents, solicitors and insurers do?
- What does the future hold if the community works together with the local authority?

The display was accompanied by a four page explanatory leaflet entitled ‘Land Stability in Ventnor and You’. A temporary information centre provided the opportunity for interested residents not only to read the display boards, but also to ask questions and discuss any problems or concerns that they had with the local authority technical staff or its consultants, confidentially if required. As a result of the success of the temporary information centre as a dissemination point, a permanent display opened in 1998 within the Isle of Wight Coastal Visitors’ Centre based in Ventnor. The town has been able to turn the geological situation to its advantage and capitalise on the interest in geological, coastal and environmental tourism and education.

The Centre for the Coastal Environment has produced a series of information leaflets that have been distributed to every homeowner in the area together with more comprehensive reports which provide a wealth of information on the range of landslide management measures that have been promoted by the Council (Moore et al. 1995). Over the last ten years, four different information leaflets have been circulated to all 2,600 property owners and a range of reports and technical information have been provided with financial support from the Council.

The European Union LIFE Environment Programme (L’Instrument Financier de L’Environnement: European Commission 1997 and 2001) has also provided financial support for landslide risk management initiatives. In particular, a LIFE Environment study led by the Centre entitled ‘Coastal change, climate and instability’ (McInnes & Jakeways 2000a; 2000b; 2002) allowed the development of the landslide management work on the Isle of Wight Undercliff to be taken forward.

As part of the Landslide Management Strategy, the feedback from local residents is regarded as very important. During the course of the LIFE project, a survey was conducted which showed that a high percentage of residents (over 60%) had lived in the Undercliff over ten
years and the majority were aware of ground instability issues at the time of moving into the area (82%). It is interesting to note that approximately 50% of those who intended to move to the Undercliff had obtained information from surveyors, consulting engineers or estate agents. The Council was encouraged to note that of those who sought its advice on ground instability, some 90% found the advice very helpful or helpful and 66% of those who responded to the Council survey had read the key report on ground movement in the Undercliff. It was very pleasing to note that all those who responded had found this report to be either very informative (55%) or informative (45%). It should be noted, however, that over the previous four years, some 25% of those responding had moved into the area, which indicated a significant turn-over in occupation of residential properties. As a result, this demonstrated the need to continue to provide up to date information for residents on a regular basis (McInnes et al. 2006).

One particular concern as far as property owners are concerned has been difficulty in obtaining insurance in certain parts of the Undercliff, often due to a lack of knowledge over the true extent and nature of ground instability conditions. Certainly the DOE study assisted the process by indicating those areas where the risk is greatest. In addition the summary report from that study (Doornkamp et al. 1991) made very positive statements about building insurance and financial development in the town of Ventnor. It was encouraging to note, therefore, from the resident’s survey that 76% of those questioned had been able to obtain full insurance, including subsidence cover, in 2003, rising to 94% by 2006. The Centre for the Coastal Environment believes that a significant contributory factor to this statistic has been the availability of better information and guidance for local residents as well as for insurers over the intervening period. The residents survey was updated in 2006 as part of the ‘Response’ LIFE project (McInnes et al. 2006) and the results provide further support for the Council in terms of the value of this kind of information for non-technical users.

Whilst the results of the various residents’ surveys have proved encouraging, it is recognised that there are still significant problems to address in terms of managing ground instability in the future. A key component of the Management Strategy is the monitoring programme, which includes recording meteorological conditions as well as analysis of data from a wide range of sub-surface instrumentation. In total, around the Island’s coastline, nearly two hundred instruments record groundwater levels, settlement, cracking and tilt. The programme benefits particularly from the climatic record which allows comparisons to be made between winter rainfall and ground instability events. Valuable historical data of this kind alongside good local knowledge forms an important tool in terms of assisting prediction, allowing the Council to be best placed to address changing conditions in terms of rainfall patterns in the future. These problems necessitate particular responses by the Council, businesses and local residents, which will be increasingly focused on adapting to these changing conditions assisted by improved educational programmes.

In Scotland, the public are potentially at risk from debris flow events on the road network (and indeed elsewhere). A key part of the debris flow management strategy described in previous sections is the DNA sequence of detection, notification and action. In turn, one of the most important elements of the action process is to inform the road-using public of events and, indeed, of the likelihood of event occurrence. This takes a variety of forms, including static signs and variable message signs, and announcements via the audio and visual media. A leaflet giving guidance to the public is in draft and it is hoped that this will be made available both on the web and at key geographic locations, such as rest areas,
around the network. In developing the strategy, the project team have taken due cognisance
of very valuable information from eye-witness accounts of the August 2004 events.

In essence, the approach in Ventnor has been both to solicit public opinion as well as to
inform, whilst the approach in Scotland has, to date, been largely an information exercise.
This is only to be expected since Ventnor is an inhabited complex and the debris flows in
Scotland largely affect relatively sparsely inhabited area.

CLIMATE CHANGE
Landslides are often cited as being caused by storm rainfall and the link between high
intensity rainfall and debris flows has been documented in many countries. However, the
influence of antecedent rainfall prior to storm events was clear from the events experienced
in Scotland in August 2004 (Winter et al. 2007b; 2007c) and other studies have made the
link between antecedent rainfall, storm rainfall and landslides, albeit that the period of
antecedent rainfall may be short or the amount of necessary antecedent rainfall may be
supplied by the early part of the storm event. Wider-ranging reviews of the different
approaches taken to the development of rainfall thresholds are presented by Anon (2007)
and Winter et al. (in press).

While antecedent rainfall has been found to be a factor in the occurrence of Scottish debris
flows, it is a more significant factor in large deep-seated landslides such as affecting
Ventnor (e.g. Bromhead & Ibsen, 2007; Ibsen & Brundsen, 1996).

The rainfall of the UK broadly increases from low levels in the east/south-east to much
higher levels in the west/north-west (Figure 16), resulting from the prevailing weather
fronts moving in from the Atlantic Ocean.

There are some indications of rising rainfall associated within anthropogenic induced climate
change in the models. The predictions are inherently variable, but generally suggest increased
winter rainfall (which is already the wet season), decreased summer rainfall with a general
increase in storminess. Predicted changes in climate patterns, including changes in rainfall,
are available from www.ukcip.org.uk. In broad terms, it is the increase in annual rainfall that
is expected to be most important in the case of Ventnor and deep-seated landslides in the
south of England, while it is the increase in storminess that is expected to be the primary
driver for changing landslide patterns in the north of the UK, particularly in relation to debris
flows.

Analyses by GeoSlope (2005), replicating the rainfall conditions experienced in California
and British Columbia in February 2005, yielded some interesting results. The analysis
confirmed that a typical model slope remained stable for seven months during which
585mm of cumulative rainfall fell but became unstable after a further 228mm over a period
of four days. Typically the failure could not be attributed to increased positive pore water
pressures as the failure surface did not penetrate below the water table. GeoSlope attributed
the failure to decreases in suction. This type of behaviour corresponds well with that
predicted from unsaturated soil mechanics theory (Wheeler et al. 2003) and the broad style
of this type of failure mechanism is supported by experiment (Springman et al. 2003).

For coastal sites, the situation is complicated further by the likelihood of sea level rise and
associated marine erosion.
Potentially these predictions would appear to be continuations of trends observed within the Ventnor Undercliff which has the longest detailed rainfall records in the country extending back to 1839. There is already a significant awareness of the potential impacts of climate change amongst many of the relatively well-informed residents of the Isle of Wight coastal zones. They have been made aware of climate change issues through the influence of the media, which has highlighted through television coverage, natural disasters such as landslides, hurricanes and the El-Nino effects globally as well as through newspaper coverage of events such as these.

![Figure 16: 30-year (1971-2000) average annual rainfall data for the UK (Anon 2005)](image)

Data presented by the Meteorological Office (Anon 1989) indicates that, in Scotland, rainfall in the east generally peaks in August while in the west the maximum rainfall levels are reached during the wider period from September to January. Although rainfall levels in the west are relatively low in August, they do increase from a low point in May. Both scenarios indicate that the soil may be undergoing a transition from a dry to a wetter state at or around August, indicating an increased potential for debris flow and other forms of landslide activity. The central area has a mix between the rainfall characteristics of the ‘east’ and the ‘west’. The rainfall peak is both lower and shorter (December and January) than in the west, but there are also small sub-peaks in August and October (Winter et al. 2007c).

Clearly, the soil water conditions necessary for debris flows may be generated by long periods of rainfall or by shorter intense storms. It is however widely accepted that Scottish debris flow events are usually preceded by both extended periods of antecedent rainfall and
intense storms.

Vegetation will also be affected by climate change. Lower overall levels and changed patterns of rainfall might be expected to increase the pressure on vegetation and thus to reduce its beneficial effect upon slope stability. Additionally, extended periods of exceptionally dry weather could potentially lead to wildfires and associated debris flow such as those described by Cannon et al. (2007).

The importance of the potential effects of climate change impacts on slope stability is exemplified by the existence of an Engineering and Physical Sciences Research Council (EPSRC) Network: Climate Change Impact Forecasting For Slopes (CLIFFS) (Dixon et al. 2006). This is funded to provide a ‘talking-shop’ for such issues and to develop collaborative working arrangements to study such impacts and to develop coping strategies.

**MAJOR ENGINEERING SOLUTIONS**

Clearly in the case of the Scottish debris flows large scale engineering interventions are not feasible, desirable or affordable on a significant scale. Indeed, the management strategy detailed in previous sections of this paper is predicated on exposure rather than hazard reduction. Notwithstanding this, the provision of enhanced drainage at the roadside and for slopes forms a vital part of the smaller scale engineering works implemented for the Scottish road network (Winter et al. 2005a). In contrast the deep-seated landslides of Ventnor are of a type that could respond favourably to a variety of engineering works and alongside the Council’s current landslide management strategy to maintain ground stability at least current levels it has been necessary to consider the possibility of undertaking further civil engineering measures, such as drainage at a scale which will meet the challenge of potentially worsening climatic conditions.

As part of a recent coastal instability study for Ventnor, a range of risk management options were evaluated in order to try and identify what additional civil engineering measures may be necessary in order to try and maintain ground stability at the current levels in the context of climate change. Following the completion of a programme of sub-surface ground investigation works, it has been possible to prepare a more accurate geological model for central Ventnor (see Figure 17). With this information, it is possible to assess risks of ground movements in various scenarios. By valuing assets that might be affected by ground movements, if no action was taken (the ‘do nothing’ scenario), it is possible to compare the losses that might be incurred against the cost of various protection or stabilisation measures (e.g. coastal protection and drainage works). With an improved understanding of the consequences of ‘doing nothing’, it is possible to calculate the ‘benefit cost’ or the economic justification of carrying out civil engineering measures. This work has been undertaken for central Ventnor and demonstrates a positive economic benefit for a drainage scheme combined with some improvements to coastal defences. Further work is necessary to confirm more precisely the nature of any drainage measures that could be undertaken; and to identify how any such project can be implemented in the future (Moore et al. 2007a; 2007b).

Previous studies have confirmed the vital role played by coastal defences in helping to secure the toe of the Undercliff landslide complex. However, although defences are in place along the developed frontages, some sections are likely to require upgrading as a result of an increased understanding of the landslide model and the implications of sea level rise, specifically the Western Cliffs frontage, which was first defended fifteen years ago.
Figure 17: Geological cross-section of Ventnor
CONCLUSIONS

The past record shows that rainfall in southern England is generally increasing. Taken in conjunction with evidence of anthropogenically-induced climate change, causing increased rainfall and increased sea levels, this leads to an increased likelihood of landslide activity. In Scotland the key climate change indicator may well be that of increased storminess, also leading to increased potential for landslide activity, albeit of a shallow debris flow type. As present levels of risk are currently perceived to be unacceptable, this leads inexorably to the conclusion that it is in the national interest to adopt appropriate best practice in management and intervention strategies for landslide risk reduction.

The work on the Isle of Wight has been ongoing for in excess of 20 years and may be viewed as a mature and well-developed approach to the problems inherent within the area. Research has been undertaken to enhance the understanding of the basic problems particular to the landslides in question and implemented within the Isle of Wight Council’s Landslide Management Strategy for the Undercliff.

In contrast, the work in Scotland has been undertaken in response to a series of events that occurred just three years ago, albeit that similar events in more remote areas had flagged-up the potential for problems in the recent past. The priority in the interim has been to establish a clear ranking of the sites most at risk based upon sound scientific principles and to implement the management strategy, developed in parallel, for the lengths of road affected. Work designed to yield a better understanding of the relationship between rainfall and debris flow events in Scotland is ongoing and further proposals are being developed to undertake further research on debris flow events in Scotland.

The spectrum of responses to landslide problems fit neatly into the ternary ‘Willingness Diagram’, illustrating in this triangular plane how different societies have different attitudes to risk acceptance, fiscal expenditure and the environment.

Climate change poses challenges to society and in the context of landslide risk management demands a response. When climate change is combined with increasing population, population mobility, economic and tourist activity, education levels and associated stakeholder activity, and decreasing acceptance of risk, there is a clear need to continue and enhance activity in landslide risk management and to develop appropriate tools and methodologies.

The examples of the management strategies for both the coastal landslides at Ventnor and the Scottish debris flows represent the state of the application of the ‘state-of-the-art’ in the UK. Although there are other areas of the UK with landslide problems, and indeed with management strategies, there remains a need for a degree of consistency of approach to be applied, to engage with stakeholders and to disseminate best practice.

REFERENCES

Anon (2005). “Causes of the weather in the British Isles.” Fact Sheet No 4 – Climate of the


and stress-strain behaviour in unsaturated soils.” *Géotechnique*, 53(1), 41-54.


**ACKNOWLEDGEMENTS**

The support of the Isle of Wight Council’s Centre for the Coastal Environment and of Dr Roger Moore (Halcrow) are gratefully acknowledged.

We would like to acknowledge inputs to the Scottish Road Network Landslides Study from Lawrence Shackman (Transport Scotland) and Forbes Macgregor (Consultant to Transport Scotland), who together with the first author managed and led the study. Transport Scotland funded the study. The Working Group for the study additionally comprised A. Forster (British Geological Survey), A. Heald (Jacobs), S. Martin (Halcrow), P. McMillan (W. A. Fairhurst & Partners), I. Nettleton (EDGE Consultants UK Ltd), J. Parsons (formerly Jacobs now Donaldson Associates), A. Sloan (Donaldson Associates), D. Spence (Highland Council) and M. Willis (Arup). The GIS-based assessment for debris flows in Scotland was undertaken by the British Geological Survey under contract to TRL with substantial input from selected members of the Working Group: the contributions of M. Harrison, A. Gibson, D. Entwisle and G. Wildman (British Geological Survey) are additionally acknowledged.
LANDSLIDE MITIGATION STRATEGY AND IMPLEMENTATION IN CHINA

Yueping Yin
China Geological Survey

Sijing Wang
Institute of Geology and Geophysics, Chinese Academy of Sciences

Zuyu Chen
China Institute of Water Resources and Hydropower Research

Abstract: China has been implementing a nation-wide landslide investigation and mapping plan since 1999, which covers more than 1,500 counties. The preliminary investigation has identified nearly 150,000 sites with potential risks. Detailed mapping projects are currently focused on the most hazardous areas. The outcomes of these investigations will include a series of maps on 1:50,000 scale as follows: landslide distributions and types; landslide susceptible zones; landslide risk zonation; contours of landslide triggering precipitations; estimation of property losses; landslide prevention plan and engineering measures. With the rapid economic and urban development, landslides associated with man-made activities and engineering developments have become a prime concern. They account for more than 80% of the landslides in China.

China has successfully carried out a number of large-scale landslide prevention projects since the early 1990s, with particular attention given to the Three-Gorges' reservoir area and the cities under rapid development in the western part of China. This paper describes the slope stabilization works for the Lianziyan hazardous rocks and the new Danba city of the Sichuan Province. The weather-based landslide forecast system has been established that covers the landslide prone zones in the nation. But, the rural areas account for nearly 80% of the total geohazard occurrence. Therefore a project entitled ‘Geohazard mitigations in thousands of villages – knowledge and actions’ has been carried out. More than 3 million people have joined the training plan and learned some fundamental knowledge in how to siting, constructing, monitoring and implementing various precautionary measures to take during emergency situations. This paper also highlights China’s state landslide prevention plan, predicting and preventing landslides by masses, i.e. by villagers living in or near the landslides, the emergency plans and risk management systems.

INTRODUCTION
China has conducted a large number of landslide reduction projects for decades under the supervision of central and local governments. The capability of combating landslides disasters has enhanced significantly. In recent years, the casualties due to landslides have reduced to about half of the peak value which occurred in 1998. The warning system of geohazard covering the mainland China has been establishing since 1999. China has successfully carried out a number of large-scale landslide prevention projects since the early 1990s, with particular attention given to the Three-Gorges’ reservoir area and the west terrain cities.
NATION-WIDE LANDSLIDE INVESTIGATION AND RISK ZONING

China suffers from severe landslide hazards every year. Landslides threaten lives and properties in 30 provinces, resulting in an estimated 700 to 900 deaths and damages of properties exceeding $10 billion RMB (Chinese Dollar) annually (Figure 1). Although most of the landslides occur during rainy seasons, human activity is the main factor triggering landslides because of the large scale construction works over a large area.

Since 1990, China has completed a geohazard mapping program at the scale of 1:500,000 that includes 55,000 landslides, 13,000 rockfalls and 17,000 mudflows, with descriptions of locations of these landslides, and the related geological conditions. The mapping program provides a large volume of information for landslide prevention works, such as those associated with oil and gas pipelines, railway, hydropower and other key projects.

More special investigations and risk zoning of landslides have been ongoing since 1999, which covers landslide-prone areas in about 1,500 counties. The key task of investigation is to identify the potential of landslides, to provide an emergency preparedness plan, and to establish warning systems for the existing villages. About 150,000 potential landslides have been identified, and 80,000 of which are monitored. This program, called ‘Monitoring and Preventing by Masses’, has proved to be very efficient for landslide mitigation. The success rate of landslide prediction and warning is obviously rising since the mapping and risk zoning plan was conducted. Figure 2 presents the numbers for evacuation due to successful prediction during rainy season from 1998 to 2005.
DETAILED MAPPING FOR HIGH RISK ZONES OF LANDSLIDE HAZARDS

The occurrences of landslides are very complicated in China. More detailed surveying and mapping for high risk zones have been conducted on the basis of geological conditions. The relationships between the landslide susceptibility of different kinds of slopes and the landslide triggering factors, such as, rainfall, earthquake and human activities, are comprehensively analyzed. This survey program is at the scale of 1:50,000, resulting in a detailed inventory and distribution maps that cover landslide susceptibility, geological and geo-environmental conditions and the impacts of construction projects, such as dams and pipe lines, etc. in the high risk zones. These documents will be used as guidance for risk assessment for urban development and relocations.

A typical case is presented here concerning the Yan’an city of Shaanxi province. Yan’an is located in the central area of a loess plateau. Incised gullies and vertical joints are very common due to soil erosions. The bedrock is also heavily weathered. Rainfalls from June to September are normally very intense. The annual precipitation is about 500 mm. Most of the geohazards are loess slides and slumps. A series of maps depicting landslide hazards have been completed as follows:

- A map of regional engineering geology conditions and zoning. It is a basic map to delineate the controlling factors and geo-environments on landslide occurrences, which include stratigraphy, tectonics, geomorphology, topography and hydrogeology. In the region, the loess is divided into three groups: Upper, Middle and Lower Pleistocenes on the basis of the geological epoch. The comprehensive factors show that the highly landslide-prone zone is controlled by the upper Pleistocene loess in the stratum and tributary geomorphology.
- A map of engineering activities. It delineates the pattern and intensity of engineering activities, such as housing, damming, road repair, mining, etc. In the region, more than 80% of the geohazards are induced by inappropriate cuttings and housing developments (Figure 3).

Figure 3: The landslide in Yan’an due to cutting which endangers the new houses
- A map of landslide locations. This map indicates the locations, scales and movement types of landslides. It is important to outline the run out of a landslide which may involve. High resolution remote sensing images, such as SPOT and Quick Bird, are widely used for investigation purposes.
- A map for landslide vulnerability zonation. This map is derived from the map of regional engineering geological conditions and the map of engineering activities, as well as the map of landslide locations. Previously, the natural geological environment is a key factor affecting landslide occurrence and human activity is of secondary importance. But now, rapid construction and large-scale cutting has become the prime concern (Figure 4).
- A map of landslide risk zonation. The work is in progress. This map is derived from the vulnerability zoning map. The risk zonation includes assessment of landslide influenced areas and intensity, as well as human activities, such as houses, schools, power plants, dams, roads, etc.
- Maps of expected property losses, landslide-hazards prevention planning, recommended relocation approaches and weather-based landslide hazards warning (Figure 5).

In China, the work on landslide hazard reduction is organized by the government at central to local levels. Therefore, it is very important that geologists provide concise information for decision-makers. The six sets of maps described above are highly demanded. Through the maps, the government can easily understand the critical issues and make decisions.

STABILIZATION AND MITIGATION OF MAJOR LANDSLIDES
Since the ‘International Decade of Natural Disaster Reduction’ started in 1990, more than 200 major landslides that severely threatened the cities, main river courses and other key public facilities, have been stabilized. As a typical example, the works at the Lianziya hazardous rock mass on the south bank of the Yangtze River in the Zigui County of Hubei Province are briefly discussed below.

The landslide is on the opposite side of the famous Xintan landslide across the Yangtze River, and is 27 km upstream of the Three Gorges Dam. Hazards of landslide have been recorded for 500 years and the most severe one stopped the river navigation for 82 years. Comprehensive measures for stabilizing the rock slope, including pre-stressed cables, reinforced concrete fillings, surface drainage and defensive dams, have been provided since 1992 (Figure 6). The monitoring results show that the stabilization has been successful in preventing the rock mass from falling into the river and threatening the Three Gorges Reservoir (Figure 7).

The Three Gorges Project is one of the largest water resources development programs in the world. The resettled population of the Three Gorges Reservoir area is about 1.2 million. During the first phase of the project from 1993 to 1997, 82,000 people were resettled, and 550,000 populations were resettled in the second phase of the project from 1997 to 2003. By 2009, over 600,000 people will be resettled during the third phase of the project. The resettlement plan is a great challenge, since it is difficult to find flat land in the reservoir area that is suitable for construction. People have no alternative but to move to landslide-prone areas. The Three Gorges Reservoir resettlement plan could be divided into three stages:

1. The first stage started before 1993, when the dam project began. The project was mainly concerned with the natural geological hazards, although some man-made slopes were included;
2. The second stage was from 1993 to 2003, which mainly focused on the toe-cutting slopes and waste rock materials;

3. The third stage started from 2003 till 2009. Landslides are expected to be triggered by reservoir water level fluctuations. Figure 8 shows the Qianjiangping landslide immediately after filling of the Three Gorges Reservoir in 2003. The third period of landslide hazard mitigation will extend to 2020.

Figure 4: The map of landslide risk zoning for Yan’an
Figure 5: The map of rainfall-based landslide risk alarming
Figure 6: Section of anchoring for Lianziya dangerous rock mass, the Three Gorges

Figure 7: View of Lianziya dangerous rock mass before filling of reservoir (photo taken in 2001)

Figure 8: The Qianjiangping Landslide after filling of the Three Gorges Reservoir in 2003
A systematic landslide prevention project has been carried out at the Three Gorges Reservoir, which is aimed at protecting and stabilizing unstable slopes and rockfall deposits and solving the engineering problems encountered in the large-scale excavation and filling carried out in the resettlement construction plan since 2001.

Another example, which illustrates the problem of massive landslides in China is described below.

Climate change has caused snow line retreating in the Tibet plateau. Events of gigantic landslides and glacial lake busting are increasing. On 9 April 2000, a high-speed huge landslide occurred in Yigong of Tibet (Figure 9). The avalanche was initiated at elevation 5200 m and deposited debris at 2200 m. It lasted for about 10 minutes and the debris traveled 8 km (Figure 10). The landslide accumulation lobe is 2500 m wide, 2500 m long, and 280 - 300 million m$^3$ in volume, and is the largest recorded landslide in the world.

Figure 9: RS image showing the damming caused by the 2000 Yigong avalanche

Figure 10: Geological section of Yigong landslide

The huge landslide created a 60 - 100 m high dam blocking the Yaluzangbu River and caused an ecological catastrophe. The reservoir water level rose by 62m in total at a rate of 2.37m per day on average. An emergency trench was excavated in the middle of the dam to prevent the water level from further rising. Within 33 days before the dam broke, 1.36 million m$^3$ meter of landslide mass was removed and the water level was reduced by 24.1 m. The dam
burst on 10 June 2000, and the water level dropped by 40 - 50m within several hours. Three billions m³ water was suddenly released, covering an area of 15 km² to 50 km². The maximum flood discharge rate was 120,000 m³/s. Thousands of landslides were triggered by the flood in the downstream area. No people died in China, due to the successful warning and evacuation arrangements. Unfortunately, when the flood went through ‘Yaluzangbu Grand Canyon’ and arrived at India side, it was reported to have resulted in about 30 fatalities, 100 missing people and 50,000 people became homeless (ICIMOD 2005).

In the western part of China, construction works have reactivated some ancient landslides. The Danba new town of Sichuan Province is located on an ancient landslide (Figure 11). Rapid expansion of the town since the 1990s has steepened the slopes. Figure 12 shows a 28 m high man-made slope that was not fully supported. The reactivated ancient landslide with a volume of 2.5 million m³ moved at a maximum rate of more than 50 mm per day. The new town would have been destroyed if stabilization works were not applied immediately. The emergency measures included several tens of thousands of sandbags with a total volume of 4,000 m³, that buttressed the toe of the slope. The displacement rate declined immediately to 5 mm per day, which indicated that the sliding had been effectively controlled. Meanwhile, 5 rows of pre-stressed cables, each with a capacity of 1,300 kN, were installed to enhance the stability of the slope against landslides.

Figure 11: The Danba town located on an ancient landslide

Figure 12: Geological section of the Danba
WEATHER-BASED REGIONAL LANDSLIDE HAZARD WARNING
In 2003, the Ministry of Land Resources (MLR) and China Meteorological Administration (CMA) signed a cooperative agreement on operating a weather-based geohazard warning service during the raining seasons. CMA provides rainfall data, and MLR will make forecast for geohazard risks, and release warning notices through China Central Television (CCTV) at prime times after broadcasting daily weather forecast. In the same manner, the local cooperative agreements have also been signed in various provinces. The weather-based warning system is part of the landslide prevention programme of ‘Monitoring and Preventing Landslides by Masses’. According to incomplete statistics in 2004, over 700 landslides were successfully predicted and warned, and 46,000 persons were evacuated from risky areas. On 24 August 2004, MRL and CMA jointly issued a warning of a class 4 - 5 landslide hazard at the Zhejiang and Fujian Province coastal area in anticipation of the Typhoon ‘Aily’ coming on 25-26 August. Two days later, the typhoon brought about 400 - 600 mm rainfall, which triggered hundreds of landslides. No people were injured or died as tens of thousand of people were evacuated timely from the risky zones, although 260 empty houses were destroyed.

GEOHAZARD RISK ASSESSMENT ON LAND-USE
The surge of infrastructure construction especially in the western mountain areas of China has created a serious concern about geohazards. According to statistics, about 80% of the landslide hazards are caused by inappropriate construction activities. In 1999, the MLR issued an act that specified a compulsory geo-hazard risk assessment for various land-use purposes before the applications of these projects are approved. The assessment includes: (1) possible geohazards induced or intensified by the project, and (2) risks of geo-hazards induced by the construction project itself.

The largest project on landslide risk assessment in China is related to the natural gas pipeline project linking the Xinjiang Autonomous Region to Shanghai. The 4,200 km long pipeline strides across various complicated geological and geomorphological areas that pose significant constraints on the pipe alignment, construction and operation alternatives of the project. The detailed review of landslides and other geohazards for this project helped to avoid many major hazards that may threaten the project.

EDUCATION AND TRAINING FOR GEOHAZARD MITIGATION
In China, development standards in rural areas are behind those in the urban areas. The rural areas are much more hazard-prone due to the low input and poor knowledge regarding landslide prevention. The MLR organized a series of training courses in 19 provinces in 2006. Three million people joined the course and learned some fundamental knowledge about safe housing and construction practices, simple means of observing and issuing warning of landslide occurrence during emergency situations, and evacuating and rescuing actions, etc. Tens of landslide disasters had been successfully warned and avoided in the raining season of 2007 as a result of the education plans.

MAIN ACTIONS ON LANDSLIDE HAZARD REDUCTION
The Chinese government has launched an action plan on geological hazards mitigation, which includes landslide investigation, risk assessment, design and stabilization of hazardous slopes, and emergency responses. The plan has also incorporated a landslide risk
management system from state to provincial levels, and is currently extending to various villages since the 1990s. A framework for landslide hazard reduction plan is suggested within the next decade. The framework includes the following key components.

**Establishment of the Legal System for Landslide Reductions**
Various items in laws and specifications will be officially specified, in conjunction with the publication of technical standards and codes by 2010. The landslide risk zonation will be a part of the state mandatory requirements in land-use and construction. Man-made related landslide hazards will be greatly reduced through rational construction and risk management. The hazard reduction strategy and planning will be an important part of China’s modernization processes, such as urbanization, the Western China development plan, Mega-lifeline project, etc.

**Improvement of Weather-based Landslide Alarming System**
The regional weather-based landslide warning system will be completed in 31 provinces for about 1,000 landslide-prone counties. In the key regions, such as the Three Gorges Reservoir, GPS-based real-time monitoring system will be established. Since landslides are widely distributed in China, the plan of ‘Monitoring and Preventing Landslides by Masses’ is still the main and efficient approach to be adopted in the next decades.

**Establishment of Professional Teams Working on Landslide Reductions**
The MLR has organized a professional system that includes the central administration and its local offices. This organization is responsible for landslide investigation, risk management, rescuing programs and community risk reduction plans. It is also their responsibility to enhance public awareness and offer training and education programs. The engineers of the teams are required to get special qualifications, receive regular training and involve in work on landslide-prone areas.

**Detailed Nationwide Landslide Investigations**
Within the next 5 years, detailed landslide investigations will be carried out in the landslide-prone zones of Sichuan-Yunnan, Qinlin-Dabashan and Hubei-Hunan where landslide hazards account for 78% of the total every year. The nationwide landslide hazard risk zoning, as specified in the mandatory technical standards, will be carried out at a large scale.

**Further Implementation of the Plan ‘Monitoring and Prevention of Landslides by Masses’**
By 2010, this network will be greatly improved. A number of hazardous areas will be equipped with advanced and automatic monitoring facilities. The real-time monitoring and warning systems on major key zones, such as the Three Gorges Reservoir, will be completed, when the reservoir water level reaches the normal operational level.

By implementing this plan, it is expected that, by 2020, landslide occurrence rates and losses of properties will reduce by 70% of the current level. The loss of lives will be 300 per year compared to the current annual fatalities of about 1000. The annual economic damages are expected to be reduced from the current 10 billions RMB to 5 billions RMB.

**CONCLUSIONS**
This paper reviews the landslide hazards prevention plan carried out in China during the
International Decade of Natural Disaster Reduction since 1990. With the rapid economic
development in China, landslide hazards are increasing. This paper has briefly discussed
various aspects related to this plan. These are the nation-wide landslide investigation and risk
zoning; stabilization and mitigation of major landslides; weather-based regional landslide
hazard warning; geohazard risk assessment on land-use and main actions on landslide hazard
reduction. These measures have proved to be successful and will be even more rewarding in
the future.

REFERENCES
Mountain Development Profile No. 10, International Centre for Integrated Mountain
(Website: http://www.icimod.org/archive/icimod/publications/profiles/10.pdf)
REPORT ON LANDSLIDE RISK MANAGEMENT PRACTICE

Andrew Malone
The University of Hong Kong

Day 1 was devoted to country reports and discussion on slope safety systems and landslide risk management practice. The session was chaired by Andrew Malone and Suzanne Lacasse and Jordi Corominas were discussion panellists. Discussion questions were framed to elicit views on institutional arrangements for slope safety and to stimulate debate about the role of the risk management process in slope safety.

Are Dedicated Government Departments with Comprehensive Responsibility for Slope Safety Needed / Feasible in Countries Seriously Affected by Landslides?

It was noted that some small territories, such as Rio de Janeiro and Hong Kong, have created specialist agencies for the purpose of slope safety management, whilst others, e.g. Andorra, have made progress without one. Malaysia appears to be the only large country with three-tier government to have set up a dedicated government department with comprehensive responsibility for slope safety.

Whilst some speakers expressed a preference for such specialist agencies, others felt they were not essential in their countries. A number of speakers said that it was imperative to get clarification of responsibility for slope safety at cabinet level. It was generally agreed that because slope safety management relies on the work of many governmental agencies (e.g. for building control, land use planning, civil defence, transport, geological survey, meteorology, works) a ministry in central government should be tasked with the duty of coordination, funding, planning, directing, monitoring and providing the framework for slope safety management.

That achieved, the next priority was felt to be the supply of adequate resources to the slope safety effort. It was accepted that resources for safety tend to come naturally only in the wake of disaster and normally the willingness to spend is transitory. It is not easy to get steady long-term funding to mitigate infrequent perils but this is what is needed. The view was expressed that skilful management of public awareness and adroit handling of the resource allocator are required in these circumstances. In these tasks, global risk (i.e. a number expressing the prospect of harm) provides an index of the overall state of safety of a city or region, especially useful during uneventful years when interest wanes, and a parameter to help measure effectiveness and thereby build confidence in the slope safety system.

How Can We Measure the Effectiveness of Public Expenditure on Slope Safety?

The question is: did your intervention work and if so, how much good did it do and did it give value for money? The question relates to a city, region or country in which a slope safety system has been operating for some time. People tend to ask such questions after a disaster. Four parameters for measurement were referred to in discussion: actual losses (a declining rate of fatalities), risk as estimated by global Quantitative Risk Analysis, improvement in public satisfaction as measured in public opinion survey and cost-effectiveness in Cost Benefit Analysis. It was suggested that a cost-effective system is one where the value of loss prevented by the system is greater than the cost of the system, or more narrowly, where system cost per life saved is less than a benchmark value of preventing a fatality.
It was explained that in judging its slope safety efforts Hong Kong has regard to the trend of the number of fatalities per year and ‘before and after’ estimates of global risk. As yet, economic loss is not quantified. Internally, there is an assessment by the government auditor of value for money. Hong Kong has also conducted public opinion survey to measure public satisfaction with slope safety efforts. It seems that the trend in the number of fatalities per year is also used as a slope safety indicator in Mainland China. Speakers suggested that the continued willingness of Hong Kong to pay for the slope safety management system (after thirty years) was an indication of the perceived effectiveness of the system.

There are difficulties with measurements of this kind. To the question ‘How can you really be sure any modification of fatalities is the consequence of your action?’ the reply was given ‘by looking at all the other possibilities and ruling them out logically’. One such ‘other possibility’ is inter-annual rainfall variability; a speaker said this is accounted for in Hong Kong by normalizing fatality data for annual rainfall. It was accepted that measurement of the effectiveness of public expenditure on slope safety was contingent on the availability of long-term data.

**How Safe is ‘Safe Enough’?**

The risk management process provides a logical framework for decision-making on safety. The procedure includes: defining objectives, identifying hazards, estimating risks, evaluating risks, managing risks and monitoring and updating. In a number of countries, including common law and civil law jurisdictions, the methodology for the regulation of safety in the vicinity of major industrial hazards relies upon the risk management process. As part of evaluating risks the estimated risk levels are judged against a risk yardstick (‘risk criteria’). Some of the criteria used are quantitative, stipulating the levels of risk of death, individually or in multiple, which are unacceptable and tolerable, or tolerable subject to conditions. One such condition is known as the ALARP concept (as low as reasonably practicable), a notion derived from the law. The question ‘How safe is “safe enough”? has been answered for practical purposes when the level of tolerable risk has been specified in the law or by the statutory regulator.

A speaker explained that an Australian Building Code Board guide of 2006 recommends the use of the risk management process (risk determined by ‘calculation’) in the statutory permitting of construction on sites prone to landslide hazard. The approach has been adopted by several municipalities. The Australian Geomechanics Society (AGS) has produced the associated guidance documents.

The Hong Kong government also makes use of the risk management process (with risk determined by calculation) on occasions in studies of site-specific hazard from landslides and boulder fall on natural terrain, nearly all of which is in government ownership. It sometimes uses the same approach in the planning and statutory permitting of developments known to be subject to significant hazard from natural terrain. But the risk management process is not part of the standard Hong Kong methodology for permitting of development involving man-made slopes, which continues to follow the rules-based factor of safety approach.

It was said that the Australian Building Code Board guide leaves the local statutory authorities to put the numbers to the risk criteria. The AGS documents suggest values to use: tolerable individual risk to life for slopes affecting residences new slope $10^{-5}$, existing slope $10^{-4}$ per annum loss of life (person most at risk). Similar values are used by the Hong Kong
government in dealing with natural terrain. The *societal* risk criteria guidelines used by the Hong Kong government for sites threatened from natural terrain are based on Figure 7.4a ERM-HK Ltd 1998, which applies to a segment of hillside up to a 500m wide. The criteria have to be scaled up linearly for wider segments. For sites with risk levels in the unacceptable zone of Figure 7.4a, measures are taken to reduce risk to the ‘ALARP’ zone. For sites with risk within the ALARP zone, risk is reduced if there are cost effective treatments but otherwise is regarded as tolerable. It was said that the Hong Kong risk criteria produce a very safe result, but one which is considered appropriate in the Hong Kong context.

It is evident that different meanings of the term ‘ALARP’ are in use, even in the same jurisdiction.

A speaker pointed out the desirability of securing public acceptance of risk criteria, emphasising that education should send the message ‘risk can never be eliminated’.

**Future Strategic Directions in Relation to the Theme of Landslide Risk Management and Practice**

Two speakers felt that the quantitative risk methodology and criteria should be adopted by law in their countries for application to slope safety. It was generally recognised that society profits from such an approach; some examples of the benefits appear above. However, it was accepted that the calculation of landslide risk is onerous in its data needs and relatively unreliable even when carried out properly. These present drawbacks need to be addressed, but they should not deter the profession from advocating use of the tool to governments, as an aid in decision-making about landslide risk.
SESSION ON LANDSLIDE RISK MANAGEMENT PRACTICE:
RECORD OF DISCUSSION

Johnny Cheuk
Department of Civil Engineering, The University of Hong Kong

H. W. Sun and Jonathan Lau
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

FIRST PART OF THE SESSION

Andrew Malone:
Can I begin by inviting discussion or comment on question 1 - How can we measure the effectiveness of public expenditure on safety? Government can spend hundred million US dollars on building a bridge. We can all see the product. We can use it. We can admire it. If we spend a hundred million dollars on safety, what can we see? How can we turn this into something tangible? So that’s the first question. Can it be done?

Suzanne Lacasse:
I guess I cannot answer the question but I can give you an example. Just before coming to Hong Kong, I was called in by a special commission in the Ministry which will make in December 2007 a recommendation on the need for coordination of landslide activities in Norway. They asked “can you tell me how much money we are going to save by doing these studies?” The politicians, or each Ministry, have to choose the causes they want to campaign for. Landslides are competing with hospitals, and taking care of old people, and having kindergartens etc. So justifying the effectiveness of public expenditure is very difficult when you are competing with the social issues. We should find a way to put this as a long term priority. And I don’t know how to convince politicians that it is as important to put money on the long-term issue as the short-term issue.

Ki-Tae Chang:
I want to mention one case – a risky highway slope in an industrial area I provided the road authority with information about the potential economic loss due to slope failure. If there was a shallow failure, one hour traffic delay will cost 40,000 US dollars, a two hour delay will cost 130,000 US dollars. So slope treatment can avoid loss of a lot of money - that was my logic, and it was successful.

Andrew Malone:
Three country reports make reference to the measurement of landslide safety improvements. These are the reports from Hong Kong, Mainland China and Korea. Hong Kong is going to reduce slope risk by 2010, to less than 25% of what it was in 1977, which is about 23% per decade. Korea is considering cutting risk by 30% in 10 years. China expects losses will reduce by 70% in twenty years, which is 35% per decade. Perhaps Raymond Chan may wish to comment on GEO’s experience.
Raymond Chan:
In HK we quantify our risk reduction achievements in terms of loss of human life. Up to now, we have not yet included quantified social and economic losses. About effectiveness of expenditure, internally, we do have some assessment in terms of the cost of each life saved. It is still a reasonable number: I don’t want to get into the debate about value of life or value of preventing a fatality. We have an audit department within the government. They did a value-for-money study on our work, a very comprehensive study, and published a report. The conclusion is that we are getting good value for what we have done. They actually recommended we should do more. That is an objective measurement from non-engineers on the effectiveness of our public expenditure. Another measurement in the public arena is the willingness to pay. Up to now, our resource allocator is still very willing to pay for our work. I did succeed, actually just a month ago, to get approval from our highest authority to get on with our post-2010 programme after our LPM programme which will end in 2010. We got approval to have a rolling programme in the future. Each year, we are going to spend HK$600M on slope mitigation and landslip prevention work. I did a press conference to announce that programme 2 weeks ago. The public reaction was very positive. We did not face any questions like why don’t we spend the money on social welfare, on education, on medical etc. We did not have a single question like that. People seem to be convinced that we are putting the adequate amount of money in the right place.

Andrew Malone:
Why do you think that you still get support though nobody has been killed by landslide in Hong Kong for quite a long time?

Raymond Chan:
It is partly due to our public education and publicity programme. We have been doing a lot in that area to maintain public awareness. Under unfavourable conditions, major disaster could happen any time. We have been telling our media reporters that, despite all our efforts over the past 30 years, our engineers have only dealt with no more than 3% of all the sloping land in HK that we can claim that we have changed their actual engineering behaviour during heavy rain. For the remaining 97% sloping land, they do not know that the GEO exists; they will behave as if they were. So the risk is there. We have been able to reduce risk to life in a big way because of our selection system, a very rigorous system. We only selected the most deserving slopes to fix with priority. Therefore, by doing only 3% of the sloping land using engineering methods, we manage to reduce the risk to less than 25% of the overall risk in 1977. The potential risk is there. Our general public is aware of it. The resource allocator is aware of it, because our public education is not only targeting the general public. We did a lot of work to educate our resource allocator that the risk is there; it is real.

Oldrich Hungr:
Landslides are geological processes, which operate on a long time scale, but operate very irregularly and randomly. So an effective landslide safety management system is one that spends less money than the annual risk cost, measured over the long-term. Unfortunately, very few places in the world have a sufficiently long database to calculate the long-term risk correctly. So that is a limitation we have to recognize. The only approach that we can follow is the one adopted by the GEO in trying to rationally assess the long-term viability and to measure the work from year to year in terms of a long range and knowledgeable perspective. We must resist emotional responses, such as, last year we have a landslide so we have to panic and spend money like crazy. Or last year we have not had a landslide, so we can cut spending. We have to somehow make this rational and sustainable.
Jordi Corominas:
I would like to add a different point of view. We are dealing with two different situations. One is for man-made slopes, another situation is for natural slopes. I think that the administration is much more willing to spend money on man-made slopes. I suppose society asks for much higher safety for man-made slopes than for natural slopes. People accept some degree of risk from a natural landslide, a natural phenomenon. So I think we should distinguish between these two situations. In fact, the case in Hong Kong is mostly related to man-made slopes.

Raymond Chan:
Just want to make a note about natural slope risk and man-made slope risk. To engineers, it is very clear. We adopt very different methods to mitigate the risk. But for politicians and resource allocator, they can’t distinguish the two. That’s my experience. In my recent presentation to senior government officers trying to get the post-2010 programme, I needed to explain the difference between the two. The risk to life is what we are trying to mitigate. For our post-2010 programme, we will call it Landslide Prevention and Mitigation Programme – LPMit Programme. I include ‘Mit’ there is simply to highlight that for natural terrain we are going to do mitigation, but not prevention. That is very different. We deliberately use this abbreviation to alert the resource allocator that’s what we meant. It’s mitigation that we are talking about. We can never prevent natural terrain landslides effectively.

Eric Leroi:
I think it’s always very difficult to quantify something you avoid. That’s not only a problem for landslide risk management, but for all the other phenomena causing losses you want to prevent. Also I think it is difficult to measure the effectiveness of public investment in countries where the level of risk, the number of fatalities and the number of landslides, are very low. A good example is Canada. With very few fatalities, even a large reduction appears not of great value. But it’s easier to use this method of measurement where you start with lots of fatalities. Then on a related matter, politicians are wary of cost-benefit analysis. None of the different political systems are fond of going this way, especially if they are responsible for the effectiveness of the measures realised. This was seen in France when the State was responsible for land–planning which has to take into account risk prevention. At this time, the State didn’t want to go too far in giving tools to measure the effectiveness of its own work. Now, because of new laws, local authorities are responsible for land–planning, the State being only in charge of controlling afterwards whether the developments fit with all the different regulations. And now the State is more keen to develop tools to measure the effectiveness on how risk is taken into account in land-use.

Suzanne Lacasse:
I would like to comment on Eric Leroi’s comment. He said France is distributing the responsibility from the central government to the local authorities. In Norway, at least NGI is working on putting responsibility in one central place. There are up to ten ministries that are involved when a landslide or a snow avalanche happens in Norway. The counties are responsible but they do not have the competence or the resources to be able to follow up. We believe there has to be central direction - the central government has to have the final word and the decision on how to plan and how to organize things. So our experience in Norway is that it is not that good to spread the responsibility for landslides.
Andrew Malone:
Do you feel you need to have a central office dedicated to slope safety, with comprehensive responsibility, like exists in Malaysia or Hong Kong?

Suzanne Lacasse:
It’s going to be a much more efficient system.

Andrew Malone:
Can I put that question to people? Who agrees with that?

Michel Jaboyedoff:
I think both Eric and Suzanne are right in a way. The first issue is the land planning. This is managed by local authorities. But it can be only managed well if the central government has set a clear framework, otherwise, you have something very fuzzy. In some places, you will have a lax arrangement, and in the other places, regulation will be more strict. We are facing such problem in Switzerland because we have a high pressure for tourism. And in the mountain area, we are selling even small chalets for millions of dollars. It creates new problems. We must have at central government level, very clear recommendations, and also databases and other facilities. But the application of the regulation is usually, in Europe at least, at the local level.

Meei-Ling Lin:
Before 1997 in Taiwan we had many different government agencies involved and they all did various things on landslide mitigation. We found that that’s not very effective, all moving in different directions. So that’s why we set up a central programme to coordinate and to plan for all kind of things that need to be done separately under each government agency. The programme is under the National Science Council, which is the authority for science and technology research, and is administered by the National Centre for Disaster Reduction. The central government and the local government have different authorities over landslide mitigation according to the law of natural hazard mitigations. But the land use planning is under the central government, because it is the best way to do it, because land is very precious in Taiwan.

Andrew Leventhal:
In the philosophy for landslide risk management in Australia, there is a lot of synergy with the dam industry and, in particular, the logic of societal risk is very similar. In dam safety cost-benefit analysis, the value of preventing a fatality is assumed to be AU$1million. Maybe a comparable value could be considered appropriate for slope safety cost-benefit analyses.

Mike Winter:
Cost-benefit analysis is of course a well established technique. The work we have been doing relates to mitigation of landslide risk to road users. Cost-benefit analysis is used to prioritize different options for road improvements. In the UK, we value a death due to road traffic accident at about 1.5 million pounds, which is just slightly above 3 million US dollars. It includes not just society impacts, in term of family lost earnings, but closure and congestion. But the point really is that if we do cost-benefit analysis in terms of how we measure effectiveness of expenditure on landslide safety management systems, you will quickly find that measures to mitigate road safety will win out every time over landslide safety management. We have a fair bit of experience in this over 15 years or so in terms of rock fall and debris flow. But there is another factor that we need to bring into this which takes us
back in the other direction, that is not so much willingness to pay but willingness to accept risk. There is a very definite feeling among at least parts of the profession in Scotland that when people get into their cars, they tacitly accept a degree of risk of road traffic accident, but don’t, when they do the same thing, accept that same degree of risk from landslide.

Andrew Malone:
Does that mean in cost-benefit analysis you should put a higher monetary value on preventing a landslide fatality on the roads than a traffic accident fatality?

Mike Winter:
That is one potential solution, but I think that more importantly there is a debate to be had about how we value landslide fatalities (and serious injuries) in the broadest sense, taking account of the way in which different societies and parts of societies perceive such risks, and not just in the monetary context.

Andrew Malone:
It would be interesting to hear about Mainland China and Korea’s plans to measure the effectiveness of their spending on slope safety. Su-Gon, you say in your paper you are considering a plan to reduce risk from slope failures by 30% over ten years. How are you going to measure that?

Su-Gon Lee:
Generally we take into account human loss and not the property losses. We are more concentrated on the human loss.

Andrew Malone:
And I think the measure in China is the losses, Prof. Yin, is that correct?

Yin, Yue-Ping:
Yes, I mentioned that the maximum annual deaths from landslides in China was about 1500 in 1998. According to an assessment by us, the natural reasons occupy about 80% and human activity reasons about 20%. But as you know since the human activities increase over the years in China so this year the natural reasons is only 20% or less than 20%. Because the village people already know something about landslide hazard, such as in Sichuan this year, about hundred landslide events have been predicted by local people, so over five thousand people were evacuated before landslides.

Andrew Malone:
So you can already see the effect of your actions in the reduction in fatalities you think?

Yin, Yue-Ping:
Yes, because through the investigation, surveying and mapping of the natural reasons we indicate the potential. We sent the ‘understanding cards’ to villages so before the rainy season the local government asked the people to take safeguards.

Andrew Malone:
Can I ask you about the fatality data? Your figure for 1998 for example is 1573 deaths. Now China is a big place with a billion people. How easy is it to get reliable fatality data?
Yin, Yue-Ping:
In China there is the Ministry of Land Resources and the Central Geological Survey, at the second level we have the provincial bodies and then at the county level we have the local professionals and technicians sent to the villages from the central agencies. Every year if the landslide occurs the local people must send a message to the central government, the central station in Beijing, within 22 hours. Such as in Three Gorges area if a landslide occurs in a rural area or town area they must transfer the message to Chongqing city and then Chongqing must send the information to central government. So I think these data are quite reliable. But for the goal to reduce the deaths by 2020 we must first train the geotechnical engineers especially about the geological conditions with landslide potential. This requires local knowledge.

Andrew Malone:
Thank you very much. Eric, have you been convinced by what you have heard that it is viable to measure the effectiveness of public expenditure in this way?

Eric Leroi:
How can you really be sure any modification of fatalities is the consequence of your action? It’s very difficult.

Andrew Malone:
Presumably by looking at all the other possibilities and ruling them out logically. Do you feel that’s impossible to do?

Suzanne Lacasse:
It is possible to do only if you have enough long term data.

Andrew Malone:
So, what about calculation of risk which is even more difficult than measuring purely in terms of counting the number of fatalities. How do you feel about using risk?

Eric Leroi:
I think it’s very difficult to do as well. In France we are carrying out only qualitative approach.

Andrew Malone:
Comment from the risk people in Hong Kong?

HN Wong:
The way we’ve been doing it is using QRA. That’s only a tool to do it, I fully agree that we still need data. Without data, I think we can get anything we want. But the point I would like to add is that when we talk about willingness to pay, whether the resource allocator is willing to give us money to do slope works, very often it’s not only a matter of consideration of the effectiveness of public expenditure. Of course, in a QRA framework, we can always calculate the dollar per life saved and then we have that compared with the maximum credible expenditure and in a place like Hong Kong where life could be expensive and where the infrastructure is very sophisticated we can easily justify that along that line. But that justification is only inside-looking because we are looking at whether the money is worth spending from the point of view of landslide mitigation alone. But in terms of competing for resources, we are actually competing with other agencies looking after other hazards, being
man-made or natural. So that’s more than consideration of whether our approach is effective or not and very often that comparison would involve many other factors. For instance from time to time we launch the argument that we have to discharge due diligence. Right? Something that we know to be risky and we are responsible for fixing it so that go beyond consideration of whether that is effective or not that’s something we have to do and very often the politicians would come in this is something we need to do and then we will get the money to do it but the other consideration is of course say in the case of Hong Kong where we have a central authority GEO looking after slope safety. Very often whether we get the money or not depending on whether the resources allocators trust us or not. Just to quote you a parallel when my son ask my wife for pocket money whether he get it or not would depend on whether my wife see that he has been behaving himself well and I think that’s the reality of life.

Jordi Corominas:
I would like to comment that my personal impression on this is that the indicators we use for analyzing how effective our measures are may not be so good. Our mountains are more crowded than they used to be. My guess is that we are having an increase in number of fatalities due to the fact that there are more people going to the mountains. The number of villages and new developments we have in the mountains are increasing. Consequently, we may expect that the total damage in properties and infrastructures will increase as well. So if we use only absolute figures we lost a little bit what's going on. We need to use also some relative figures in the sense that, for instance, even though we may reduce the losses per unit area, if we have an increase in the activities in the dangerous places, then we probably will have an overall increase of risk. This is something we have to take into account. On the other hand, the term risk may be misleading as low risk does not mean low hazard. We might not be taking preventive measures in places where hazard exists, but there is no or little human activity, so the risk is low. If you let these ‘low-risk’ places develop you will increase the risk. However, it is difficult to convince the decision-makers to take any action before people go there.

WK Pun:
I would like to go back for the question of whether we could account for the effects of say rainfall and other variation in the consequence of landslides. Actually this question also puzzled us for some time because we have been trying to measure the effectiveness of our system. Different years have different rainfall and so how could you account for that, to check if reduction in landslide fatalities is due to our efforts or if it is due to reduction in rainfall. So fortunately in Hong Kong we have much rainfall and landslide data, so we have devised some scientific methods to try to normalise the effect of rainfall. One simple method is based on long-term data on rainfall and landslide; we establish a correlation between the rainfall and landslides. So every year given a certain amount of rainfall we use this correlation to predict the number of landslides that would occur and then to compare with the actual occurrence of landslides. And when we have a sufficient number of years then we can see whether there is a trend of improvement. If the ratio of the actual number of landslides or fatalities compared with the predicted one regularly decreases, then this indicates an improvement.

Raymond Chan:
HN brought up a point of winning public trust on our work and one of our very effective way as far as I see it, is we demonstrate to our resource allocator and the general public that our work has been properly benchmarked with international practice. To do this we have an
international technical review board on our slope safety activities and I think that helps to win trust to some extent.

_Eric Leroi:_
Maybe you can prove the effectiveness of your measures if public expectation is satisfied? In fact, if you don’t have any reaction from the population, whatever the level of investment you put, it could be enough. If you have reaction from the population, whatever the level of investment you can have, it won’t be enough.

_Oldrich Hungr:_
One challenge in proving the effectiveness of slope management work is that the people who are exposed to landslide hazard are a very small fraction of the population. So it’s difficult to use average measures, average indices, to compare the effectiveness because it doesn’t affect the average person. In Australia for example; I have not been to Australia, but imagine there not many people are really seriously affected by landslide hazards. But I think that the society does, we as humans, feel a need to help those who are unlucky. And so if for example we said on average the Hong Kong landslide safety measures are not very significant and so we will not continue them. That basically, could be a death sentence for certain people who live under dangerous slopes. I think that we may not be able to make all of these decisions completely rationally. Psychology, may be, has to figure in as well.

Andrew Malone:
Thank you very much to all of the people who spoke this morning and to all of those who contributed to the discussion.

SECOND PART OF THE SESSION

Andrew Malone:
We know in a number of places - Malaysia, Hong Kong, Rio - there are dedicated government departments with a comprehensive responsibility for slope safety. Perhaps other places might benefit from such an arrangement. Question 2 is: are similar departments needed / feasible in other countries badly affected by landslides? Is there any possibility that this may happen in France - a single office charged with natural risk management?

_Eric Leroi:_
I think with the French Ministry of the Environment we have such a kind of office which has a lot of responsibilities regarding natural risk management. I think it’s not a question of whether you have one specific department for managing risk. It depends on the means you give to this department Because you may have all the responsibilities if you don’t have any means, I showed the slide comparing the means between Hong Kong and France, if you have a ten times less money to solve the problem even you have all the responsibilities, even if you are well organized, I think you will be facing a lot of problems. So you need first of course a good organization but after that you will need the means.

Andrew Malone:
Resources follow disaster and the responsible office would be the first people to get the money.
Suzanne Lacasse:
I would be definitely in favour to having a central office and I just can confirm that resources follow disaster. We have experienced this for snow avalanches. We have quite a bit of resources for the potential rock slides in Aknes - they came after considerable effort to convince the ministers and the prime minister that there was a big hazard there, and there was quite a bit of newspaper action there that convinced them that the money has to be made available and not that much amount of money either. But having said that there is no way that we know that one rock slope is the most dangerous in the whole area there are other problems that might have higher probability of failure we don’t know.

Andrew Malone:
So you mean you might have to wait for the disasters to get the support?

Suzanne Lacasse:
To get more support, yes! For landslides in quick clays, the programme that I presented started after the last major landslide in Rissa. The same happened with snow avalanches. There was no money available. I don’t remember how many, but several young soldiers were killed while doing military exercise and same money came a few years later.

Andrew Malone:
Any other comments on this first point of discussion?

Jordi Corominas:
I would like just to mention the case of Andorra administration. Of course, resources follow the disasters. But I think they are not happy to take such responsibility because it is a small administration. With the first hazard maps in the early 90's the administration officers started delivering building permits based on them. There was a lot of argument because they had to go to very detailed and technical issues for which the administration needs many human resources. With the new hazard map at 1 to 5000 scale and the building code prepared in 2001, they have transferred the responsibility to the technicians that prepare the design. This map delineates areas with different hazard levels and the actions to be undertaken. In case that either the stabilisation of the slope or the protection of a building is required, then it is a matter of the engineering design. The administration does not want to go into these details. To deliver the permits a registered professional is required to be in charge for the design of the remedial measures. The professional is aware of the danger and must prepare the appropriate design to mitigate the risk. It is the responsibility of the designer not of the administration. The administration takes care that in areas with moderate hazard something has been done by the appropriate professional. And that is all I wanted to say.

Andrew Malone:
So obviously in the Isle of Wight, there is already such an organization with sole responsibility, clear cut and you’ve been working for twenty years or more so what do you feel about that, Robin?

Robin McInnes:
Well, the Isle of Wight being an island and also being a unitary authority with responsibilities for local government including planning, coastal engineering and risks perhaps it’s easier to have that kind of organization. Because of the problems of landsliding, coastal erosion and flooding, there’s been one unit established since 1988, in fact, dealing with all aspects relating to natural hazards. And that seems to have worked well but we have a particular
problem that is not typical of other parts of the United Kingdom. So it’s a model that suits us very well.

Andrew Malone:
Do you find the resources follow significant episodes of movements / events?

Robin McInnes:
Yes, for example in the winter of 2000/2001, well it wasn’t the wettest winter, we have rainfall records going back to the 1830’s and this was in fact the 20th wettest winter, but it produced damage amounting to 20 million pounds. So very significant for us because our normal estimated landslide damage costs are about 3 million pounds a year. But certainly as a result of the seriousness of the episodes we were able to draw in additional funding from central government.

Ashaari Mohamad:
Yes, just to clarify a bit on what we are doing. We are the major department looking after slope safety but we work together closely with other departments for example the geological department, the meteorological department and related departments on slope engineering. So we head the team and work closely so we don’t have anybody say that they are not involved. That’s the structure that we have in Malaysia. This is more like working for the country rather than just our department.

Michel Jaboyedoff:
For Switzerland there are only one or two geologists at the federal level. This means that it’s mostly private companies doing the work. On the level of the Canton, for instance, I am now in an expert group in Canton Vaud, we have no geologist at the level of Canton and we have to find one now just for the project. And we hope that we will be able to keep him after the four years of the hazard mapping. This is quite astonishing in Switzerland. We do not have a lot of geologists and you must know also that there are 26 Cantons which means 26 regions so you have 26 ways to do hazard mapping that have been accepted by the federal state. But you can do what you want and after it’s at the level of the municipality. If one municipality wants to make its own maps they can do that because the law authorises that. It’s quite complicated with the different levels in Switzerland. Canton has the decision on the method but the responsibility is with the municipality.

Eric Leroi:
I don’t necessarily agree with putting responsibility for developing risk maps with local decision makers. It’s not too good a solution for this reason; local decision makers are under pressure from local population. They are elected and so under pressure and it’s very difficult for local decision makers to say I will block this area and you won’t be allowed to do anything in that part of your land. It helps the local decision makers to manage land if they have a kind of external referee. It’s useful for the decision makers to say, it’s not because of me it’s because we have a national standard developed by the State and which obliges me to act in this way. A national authority, which could be a department, can help local decision makers to work. That’s what we have in France, and I think it makes a good balance.

Lars Blikra:
I am following up the comments from Suzanne from Norway. I am quite convinced that we really need a central government office to take care of this topic. In Norway there is at the time being quite a lot of work going on in the ministries, and it is likely that one ministry will
take care of these topics in the future, although there might be followed up or implemented by two government bodies. I'm quite sure this set up is really important, at least for the specifications or the methods for hazard assessment and risk and also ensuring the quality of these kinds of studies. Although most of the work will be done at the consultant level. An issue which Michel was touching on, you need to make sure that this body is strong enough and that they are able to ensure the quality of the work performed by the consultants. So that's a challenge and also it's a challenge to ensure you have good communication down to the county and municipal level, to the people who are dealing with the problems, rather than being remote experts or bureaucrats. So that’s the problem with having one office. You must come close enough to the people who have the problems. Thank you.

Paolo Canuti:
In Italy we have had for two years some dedicated centres of competence for flooding, seismicity and the landslides. And I think that, anyway I hope that we will have more continuity in the evaluation of hazard conditions, vulnerability and especially the monitoring. So I think it would be a good thing to have a dedicated centre.

Oldrich Hungr:
In Canada most landslides as Jean said occur along the transportation routes and so the Ministry of Transportation and Highways of the provincial government is responsible for construction and maintenance of roads and so they naturally have to contend with landslides and they have the responsibility for landslide risks on highways. But they also exercise that responsibility in small communities. They have a branch that deals with stabilising slopes, mitigation works protecting bridges from debris flows and those people also provide a service to small communities for hazards. They supply so called ‘approving officers’ who are responsible for building permits. So that’s a good system because that organization already has a very well organised network. It covers the whole territory and they have knowledgeable people. So I think for a large country like Canada that’s a good system.

Suzanne Lacasse:
Just to add on Canada, we worked for the Quebec Ministry of Transport. This Ministry has responsibility for landslides. After the deluge in 1995-1996, the Ministry created a group which has special responsibility for response and mitigation of landslides in clay.

Eric Leroi:
Of course such a department, a specific department dedicated to natural risk prevention, is useful but I think we need as well a referee that would be above such a department. When you speak about sustainable development, constraints coming from landslide risk are amongst all the different ones you might have on land. And if you apply together all these different constraints you may have land which would have a lot of difficulties for developing. And making a sustainable development means that you have to make political choice between development and protection. You need somebody above all the sectoral specific departments, dedicated to risk prevention, waste management, water pollution, protection of environment ... to make such a political choice. So I think that if such a department is very useful it’s not enough, as it has not to be powerful. Politics has to stay above technics.

Luciano Picarelli:
I can probably add something to what has been said by Paolo Canuti regarding the Italian situation which is quite structured. It can appear quite strange but the Italian situation is quite structured because after the terrible flood of Florence in 1966, a commission was set up by
the Ministry. And they worked very very well. After the work made by this commission, headed by Prof. Giulio De Marchi who is the Professor in hydraulics and construction, a law was promulgated in 1989, so 23 years after because of complicated political situations. According to this law, the Basin Authority has been established, the Department of Civil Protection has been established by law and other initiatives were being set up in particularly the basin authorities. Every Basin Authority has the power within a hydrographical basin to establish the different risk areas and this covers all of Italy. The territory is subdivided into different risk areas and the risk has been evaluated following very simple classification into four classes, obtained of course following a qualitative approach a little bit similar to the one which has been illustrated by Suzanne Lacasse and also by Jordi Corominas. At the highest level of classification, for the highest risk, it is prohibited to build anything in the risk areas. The Basin Authority also has the power to stimulate procedures and initiatives for risk mitigation in the areas of their competence. This is for the spatial analysis of the risk over the territory. Regarding the temporal prediction of landslides, functional centres have been established in all the regions in Italy and these functional centres are coordinated by the Department of Civil Protection and they send off the signal or alarm when intense rainfall is coming, and finally, the rescue activities which are carried out by the Civil Protection Department and by the municipalities. When a national emergency occurs as in the case of Sarno a delegate commission is appointed by the Prime Minister with power to aid the population and restore the situation to prior to the event, so they can launch stabilisation works and so on. Lino Versace may have something to add, he was the vice-delegate commissary for the Sarno event in 1998 so he has a good experience on that.

Andrew Leventhal:
The situation in Australia is that slope safety responsibility is partly at the local level, partly at the state level and partly at the federal level. Residential development is regulated by local government, who should know the challenges of their local area. The first exercise is susceptibility mapping and to undertake those exercises for residential and sub-divisional development normally requires expertise which would not lie with local government. This means that they engage a consultant firm to do it. What leads from this is the issue of funding. There are potentially situations where the national disaster mitigation programme would, or perhaps should, provide funding for such regional mapping – though that actually has yet to be tested. Infrastructure is generally a state level issue involving government instrumentalities such as a roads authority or rail authority. Importantly, such state instrumentalities will normally have a greater budget than a local government body and therefore have the ability to develop and run risk management systems. They hopefully operate a landslide risk management system these days. Certainly, the NSW Roads & Traffic Authority manages their many thousands of kilometres of roadway cuts and fills in that manner. I think that’s generally the case, certainly in New South Wales, and most likely the case in most of the other states of south-eastern Australia. Federal government involvement is of a strategic nature or involves the provision of funding within a broad framework. The federal authorities are unlikely to be involved in the detail of slope management for specific cases. The National Disaster Mitigation Program (NDMP) sets strategic objectives, funding is also approved at the federal level, but the funds themselves are a tri-party arrangement (federal, state and local level). The recent publication Natural Hazards in Australia by the federal government instrumentality Geoscience Australia identifies risk analysis requirements. This is an example of a strategic exercise. Disaster situations within the country are managed variously by either state or local government agencies, such as by a state emergency service e.g. 1997 Thredbo landslide that killed 18 people was managed during the disaster stage by NSW Police Service, who called on other assistance as it was needed. In 2006 the Australian
Building Codes Board published an advisory document on landslide risk management, drafted by AGS. This is an overview document that accompanies the Building Code of Australia. In regard to landslide management, within an area identified as a landslide hazard zone, risk has to be determined by the methods of AGS (2000), which is now superseded by AGS (2007). The responsibility for setting targets in terms of tolerable risk must stay with the regulator - and I don’t think anybody in this room would suggest otherwise. Geotechnical practitioners can do the risk assessment, but, pragmatically, the regulator must assume the role of establishing the tolerable risk levels. One of the reasons for that is the transfer of risk. A home-owner may decide to accept a risk above tolerable level with which he/she is willing to live. That might be well and good until a landslide occurs, or alternatively until he sells the property. The transfer of risk to the next property owner is an important consideration. Therefore, I think it’s very important that the regulator recognises that particular scenario and undertakes responsibility to determine tolerable risk levels.

*Andrew Malone:*
That leads us nicely on to the next question - How safe is safe enough?

*John Wrigley:*
I am a regulator in Hong Kong with the responsibility for risk management of dangerous goods installations. In Hong Kong potentially hazardous industries must demonstrate compliance with risk criteria. Now other industries like rail and landslide are adopting similar risk criteria. I was very interested to pick up in a couple of the presentations reference to risk criteria. For major industrial hazards in Hong Kong we set a risk target of $10^{-5}$ individual risk.

*Andrew Leventhal:*
The values of tolerable individual risk recommended within AGS (2007) for Australia are $10^5$ per annum for new slopes and $10^4$ per annum risk to life for existing slopes in the context of residential dwellings. Other levels are recommended for structures with different importance levels. This involves recognition of acceptance by the community of its tolerance of greater risk levels for an existing slope. However, once an owner modifies that slope, then it should be considered as a new slope and therefore the recommendation for tolerable risk is $10^5$ per annum. These are all in terms of the individual most at risk. Acceptable risk to life levels are an order of magnitude less. AGS (2007) also includes advice on tolerable risk to property. The risk matrix recognises consequences of landslide scenarios e.g. the extent of potential damage, and associated on-costs as a proportion of market value, rather than absolute dollar value.

*Jordi Corominas:*
I would say that there is no specific acceptable risk level for Andorra. Risk management policies have developed in the opposite way. The Administration was confronted with a reality which was that people were living in dangerous places because there is a lack of space. In the Principality of Andorra, most of the areas where people live are affected by some kind of hazard. They are threatened by floods, rock falls, debris flows, snow avalanches; they have many different hazards so most of the territory is affected by them. The first studies on natural hazards showed that most of the urban areas were located in hazardous places. The Andorra administration started by doing risk mitigation and protection works because they had the pressure of the population that wanted them to do things. So they did. In the capital of Andorra la Vella, which is affected by rock falls, the risk expressed as the annual probability of loss of life, has been reduced from $10^{-3}$ to $10^{-5}$ thanks to the construction of rockfall fences. The key issue is that if you try to calculate the risk for very large events for which we don't
know the return period, but probably it's of thousands of years, then the risk may be unacceptable because the consequences will be very high. However, taking risk management decisions based on such rare events and with such uncertainty is really a challenge.

*John Wrigley:*
I think it’s a good start with individual risk but that’s not the complete picture because it doesn’t capture what we call societal risks. The slide shows the societal risk guidelines adopted by the Hong Kong Government (Figure 1). The important point to note about this graph is that the farther you go along the bottom axis with 1, 10, 100 fatalities then the acceptable frequency becomes smaller and smaller (i.e. $10^{-5}$, $10^{-6}$, $10^{-7}$, etc.). And in Hong Kong we say any accident which can kill more than a 1000 people is unacceptable regardless of the frequency. That’s Hong Kong situation for potentially hazardous installations. And I think the Geotechnical Engineering Office in Hong Kong has moved in the same direction. If you are actually using these concepts, that is then becoming very good risk management.

![Figure 1: Interim societal risk guidelines adopted by the Hong Kong Government for natural terrain landslides](image-url)
Farrokh Nadim:
But I think a more interesting discussion is if this ALARP principle is anything useful. Because if 90% of the cases that you analyzed we end up in a tolerable zone. Basically it says do what you are doing all the time, use your money the best possible way and I know that, for example many of the people working with risk in Germany, they said this whole concept is useless.

John Wrigley:
The ALARP principle has been very useful in Hong Kong in getting real safety improvements. If you operate a potentially hazardous installation and your risk level lies within the ALARP region i.e. below ‘unacceptable’ but above ‘acceptable’, then the regulator requires you to reduce risk until the cost becomes grossly disproportionate to the benefits of improved safety.

Andrew Malone:
The ALARP duty came out of case law in England and is now in the UK health and safety statute. As one element of his checking the safety regulator uses the gross disproportion test. Regulators in some other jurisdictions of the common law world also use this test, as John indicates. The German law system is the civil law tradition. Perhaps the ALARP test is not part of the law, or if it is, it means something different.

Raymond Chan:
I would like to make a remark about these acceptance criteria with regard to landslide risk. Actually we are still calling it an interim risk criteria. I don’t think we can establish any risk acceptance criteria without public debate. And actually the Hong Kong criteria for landslide risk have never gone through any public debate. We needed the criteria at the time when we started to use the quantitative risk assessment tool to manage our landslide risk. And we borrow this more or less from the chemical industry or from the potentially hazardous installations criteria. We still call it an interim guideline and now it seems that it’s well accepted by the society, by the practitioners, by everybody. We have virtually established de-facto acceptance without public debate. However, for individual cases, even if theoretically we can justify within the acceptable region, but if a disaster actually occurs out of that case and kills somebody, I am sure that the community won’t find it acceptable. Acceptance all depends really on whether there are any unfavourable outcomes of any particular case. I think that’s reality. I think if you ask an individual they would tell you that they expect 100% safe. I am not offering myself to accept any risk. That’s the normal reaction from the general member of the public. This is involuntary risk anyway. They are not willing to accept any risk. That’s why it makes public education so important in order to manage public expectation or tolerability on slope safety. That comes to the third question really on how to manage risk. I think we need public education. We need to provide the public with the necessary information to enable them to understand the reality. Many media asked me, are we in a position to make landslide risk zero? I say that never, and I guarantee that there will be fatality in future but we are doing our best to minimize the probability. That’s how I am answering the media I don’t want them to have a wrong expectation that there will be a time when we can achieve zero risk. That will never happen in Hong Kong or anywhere in the world. Thank you.

Oldrich Hungr:
What seems to be sometimes left out is that this criterion is somewhat scale dependent it depends on the size of the area that’s dealt with. In Hong Kong they recognise that by
associating this diagram (FN curve HK interim landslide risk criteria) with the standard consultation zone which is 500 m long. And obviously I think something like that has to be done because if you take a given point on the acceptable line, let’s say 0.001 for a single fatality, this would obviously be acceptable for the whole city, for example. But it is not acceptable for a single house because it would exceed the acceptable individual risk criteria. So there is a scaling exercise that needs to be included which is sort of never talked about.

*Rainer Poisel:* 
I don’t understand these black lines (see Figure 1) as they are not logical, because there should be black lines horizontally also. It’s only a threat by numbers according to my opinion. 1000 fatalities are horrible but otherwise it gives the same risk. There should be horizontal black lines also.

*John Wrigley:* 
There is a horizontal line at $10^{-9}$ which is the bottom of the chart. But that’s quite a tough guideline!

*Suzanne Lacasse:* 
There is a challenge for the regulators. What do you do in locations where houses are already on or close to a natural slope close to failure, and the people are not willing to move? The regulators need a criterion for that too.

*HN Wong:* 
I will go back to your question on how safe is safe enough. We’ve been using the risk guideline for some time. And my experience from the technical perspective is that, when that is applied to evaluation of landslide risk that is normally very conservative. So in a way as a goal post, from a technical perspective, it is very safe. The example being in the Hong Kong context, we are not really very risky compared with many other countries. Talking about the level of expenditure maybe we’ve been putting into slope safety, the guideline can still justify the expenditure. But the biggest problem I found is not with the goal post, the problem lies with whether the assessed risk, the F-N curve I draw, is reliable. If I grossly underestimate the risk then the problem would occur, but the problem would not lie with the goal post. The problem comes when there are uncertainties, when there are mistakes in the risk assessment. So I don’t think the guideline itself would solve all the problems, it’s only part of the problem but as a goal post it’s fine.

*Andrew Malone:* 
We are getting to the end now so perhaps just a few final thoughts.

*Michel Jaboyedoff:* 
Thank you I agree completely with Oldrich that there is a problem of scale when you are using F-N curves. I try to teach use of QRA to students and I was looking for a good paper and it was really difficult to find a summary of that and other matters. For example, you might add aversion to the diagrams, sometimes it’s presented, sometimes not. That makes things more complicated and how you do the integration from F-n (small n) to F-N (large N) it changed things. Then you get some power laws which are quite different. I think that we need somewhere a summary report that summarises how to use F-N curves. We see many ways to use them and they are not always used in the correct way. I don’t know if you agree with that. And I will just end by a remark that is again about the situation in Switzerland. In Switzerland for the ‘how safe is safe enough’ question the upper limit we consider for the
dangerous process is 300 years return period. We don’t consider return periods above that, say if it’s 1000 years. We are quite adventurous, we live at risk. In the Alps we face a lot of events and the people are used to living there. As I said in my presentation, we need something to manage that situation because if we continue to build houses in the Alps and if we wish to be safe we have to evacuate everybody - that will be the biggest issue. Another issue is how to inform local people about risk, and when you have tourists, that is even more difficult.

**Eric Leroi:**
If we want to know what ‘safe enough’ means I think we have to compare risks with current risks in normal life. That’s what has been proposed to the Ministry of Environment, within a study dealing with seismic risk on existing dwellings. The minimum probability of dying in normal life in France is around $10^{-4}$ and it was considered as a criterion to say what is above $10^{-4}$ is intolerable. And then, we defined what was acceptable by saying acceptable risk has to be negligible compared to real world risk. So we took one percent and proposed $10^{-6}$ for the acceptable limit. We had a case in the Alps where a snow avalanche killed people; the probability that such an avalanche could reach those people was very low, and certainly much lower than $10^{-6}$. The mayor was still considered responsible by the judge, which was difficult to understand when you consider the risk in normal life ($10^{-4}$). If you don’t adopt quantitative risk criteria (whatever the value) the local authority will always be responsible, whatever risk reduction it achieves by protection works, by zoning, and so on.

**Andrew Malone:**
On that note we will draw the session to a close. We have had a full afternoon. Thank you very much to all of the afternoon presenters and to all of those who contributed to the discussion.
Session 2  Forensic Landslide Investigations
THE INVESTIGATION OF THE AZNALCÓLLAR DAM SLIDE FAILURE

Antonio Gens and Eduardo Alonso

Department of Geotechnical Engineering and Geosciences
Universitat Politècnica de Catalunya, Barcelona, Spain

Abstract: The paper presents a summary of the investigation carried out on the failure of Aznalcóllar dam, a catastrophic slide that caused one of the worst environmental disasters in Spain. The account of the investigation includes the examination of the field observations before failure, a description of the geometry and features of the slide, a study of the properties of the foundation clay and of the pore water pressure present in the foundation as well as a number of analyses of the failure and of the post failure dynamics. It is shown that the main causes underlying the failure were the occurrence of progressive failure in the brittle foundation clay and the presence of very high pore water pressure in the foundation. The mechanism of the failure also explains the large post-failure movements (more than 50 m) that were responsible for the large spill of tailings.

INTRODUCTION

The Aznalcóllar tailings dam failed catastrophically in April 1998, causing one of the worst environmental disasters in Spain ever. The dam was part of a large open-cast mining complex that had been in operation for decades in the vicinity of Aznalcóllar village in the province of Seville, Spain. The tailings lagoon, which has an irregular hexagonal shape in plan view, was founded on a deposit of marine clays having a thickness of not less than 60 m in the centre of the lagoon. A plan view of the tailings deposit is given in Figure 1. A perimeter dam of increasing height was built over the years as the volume of tailings increased. The figure shows also a representative cross section of the dam prior to the failure.

The confining embankment was conceived in the original design as a “downstream” construction rockfill dam. The embankment was built on top of a thin (4 m) upper granular alluvium overlying the marine clays (Figure 2). An upstream blanket of Quaternary clay, covering the slope of the rockfill and connected to a shallow diaphragm wall, was designed to ensure the imperviousness of the embankment. As shown in Figure 1, the lagoon was divided into a larger Northern part and a smaller Southern one. An inner embankment or “jetty” was built to separate the two lagoons. Coarse pyroclastic tailings were mainly deposited in the Northern lagoon while finer pyritic slimes were deposited in the Southern lagoon. Tailings have been deposited in the lagoons since the beginning of 1978. The height and downstream extension of the embankment increased continuously for 20 years, as the accumulated volume of mine tailings increased. A safety evaluation was carried out in 1996 associated with a modified design that involved raising the height of the lagoon by about 2 m above the original design.

Sometime during the early morning of the 25th of April, 1998 (when the eastern side of the embankment had a height of 28 m above the foundation), a failure took place involving a substantial section of the confining embankment. As a result, several millions of cubic metres of highly acidic liquefied tailings poured into the Agrio and Guadiamar valleys. A
twenty-four kilometre length of the Guadiamar River was affected by the mudflow. Figure 3 shows an aerial photograph of the breached embankment, the inundated valley and the partially emptied and eroded tailings. The authors were expert witnesses in the judicial case that was initiated immediately after the failure. Once a final judgement had been delivered, the authors received authorisation to publish their analyses and conclusions. A detailed account is presented in Alonso & Gens (2006a) and Gens & Alonso (2006).

Figure 1: Plan view of the Aznalcóllar tailings deposit and representative cross section of the failed dam.

Figure 2: Cross-section of the tailings lagoon. Two stratigraphic discordances are observed: A lower one (high angle) between the Paleozoic substratum and the upper Miocene deposits; and an upper one (low angle), between the Miocene clays and the Quaternary deposits.
FIELD OBSERVATIONS BEFORE FAILURE

Following the recommendations of an evaluation of the dam safety, performed in 1996, a monitoring programme was set up. Figure 4 shows, in plan view, the position of the instruments along the length of the embankment later affected by the failure. Settlement plates, piezometers (in fact observation wells) and a few inclinometers were located along the crest of the embankment (a few fixed points, shown as PF in Figure 4, were used as a reference for plate levelling). Great significance was assigned to piezometer readings and a safety procedure was established in the event of piezometers indicating a rise in water pressure or in the case of abnormal outflow. The “piezometers” were slotted open tubes penetrating 2 m into the foundation clay. It is clear that they were not intended to measure pore water pressure in the clay, but rather any water pressure developing in the granular alluvium or in the pervious rockfill.

A visual inspection of the dam was also routinely carried out. A few hours before the failure (in the afternoon of April 24th, 1998), an inspection was conducted and the observed state of the dam recorded on an appropriate form. Nothing untoward was detected.

Inclinometer I-3 was located approximately at the centre of the section of the embankment that slid towards the river at the time of failure. A few readings were made during December 1996 to December 1997. They are shown in Figure 5. The locations of the rockfill-alluvial and alluvial-clay interfaces are shown in the figure. Unfortunately, the upper part of the inclinometer was damaged in January 1998 and no data for the months immediately preceding the failure are available. The displacements shown in Figure 5 were not interpreted, at the time of readings, as a sign of a possible deep sliding. It was later established, after the failure, that a sliding surface developed at elevation 26 m. Six millimetres of displacements accumulated at this elevation during the year 1997. Most of the observed displacements were recorded within the rockfill embankment, which was under construction.
Figure 4: Position of monitoring instruments, southeastern side of Aznalcóllar dam

Figure 5: Readings in Inclinometer I-3 in the direction perpendicular to the dam axis (towards the East) (For readings from Jun 97 to Dec 97)
GEOMETRY OF THE FAILURE

It became clear soon after the failure that a translational movement was the reason for the breach that opened in the embankment. There was no evidence of other possible mechanisms, such as overtopping of the embankment, instability of the embankment slopes or internal erosion. The investigation first considered the geometry of the slide, then the strength properties of dam, tailings and, especially, the foundation, were established. Finally, a consistent geotechnical model able to integrate the measured properties and field observations has been devised.

The breach in the embankment immediately north of the jetty, which divided the Northern and Southern lagoons, was a direct consequence of a deep translational slide, south of the breach, which displaced 600 m of embankment and its foundation towards the East. The failure surface was located inside the blue clays. The displaced mass included the embankment, the alluvium terrace and about 10 m of the blue clay. Figure 6 shows the cross section of the slide at the position of Profile 4, defined by boreholes S4-1, S4-2 and S4-3. The boreholes located upstream of the embankment provided a precise position of the failure surface since the tailings were found in direct contact with the clay in some of them. In a number of boreholes, it was also possible to identify the position of the sliding surface where a highly polished surface could be found. This is the case for the plane shown in Figure 7.

Figure 6 indicates the folded structures found at the foot of the downstream slope of the embankment. The interpretation given is based on zones of intense shearing found in boreholes and trenches. Upward displacements of the ground, close to 8 m, were measured at the position of the folded strata. Figure 6 provides a reconstruction of the original position of the sliding surface before failure displacements. An interesting finding of this reconstruction is that the head scar of the slide was a near-vertical surface located at the original upstream toe of the embankment. This vertical scar was fairly well preserved even after the intense erosion of the tailings deposits, which followed the uncontrolled flow out of the lagoon. Observations made a few hours after the failure, showed small mud volcanoes on the solidified tailings upstream of the dam. This was a clear indication of tailings’ liquefaction, during the failure process.

Figure 6: Cross-section of the slide at the position of Profile 4. (a) Geometry after the slide as interpreted from borehole results and surface topography, (b) Reconstruction of the position of the sliding surfaces before the failure.
The inclination (1.5°-2°) and the planar nature of the basal sliding surface is a strong indication that it followed a bedding plane. The central part of the embankment moved 40-55 m towards the East (directions varied between 93° and 100° with respect to North). The magnitude of the displacement decreased to 20-22 m at the northern limit of the slide, at the position of the open breach. Towards the South, a more gradual decrease of displacements was observed. In total, a 600 m long portion of the South-eastern embankment of the lagoon was affected by the slide. North of the breach, no displacements of the embankment were detected.

Figure 7: Highly polished and striated shear plane that follows a bedding plane (So), at a depth of 33.8 m in Borehole 2.1. It was interpreted as belonging to the sliding surface.

GEOTECHNICAL PROPERTIES OF THE FOUNDATION CLAY

General
The detailed stratigraphy of a portion of the blue clay, from Borehole S3-1, is shown in Figure 8. The sliding plane was estimated to be located at elevations 26 -27 m. In one location, a warped bedding plane was identified. Subhorizontal laminations (dip: 1°-2°) presumably parallel to bedding planes were also observed. Shear bands were also detected at non-regular intervals. They are often inclined at a significant angle. Pyrite micro nodules are scattered throughout, often showing linear arrangements parallel to bedding or lamination planes.

Some samples taken from boreholes located outside the slide area were also tested in order to evaluate the homogeneity conditions at a larger scale. Additional tests were performed in block samples taken from the large clay blocks deposited by the mudflow in the debris fan adjacent to the dam breach. The samples tested exhibit a high percentage of clay sizes (< 2μm) using standard sedimentation techniques. The clay fraction varies between 47% and 58%; average of 53%. They are classified as MH or CH (wL = 62% - 67%; Ip = 31% - 35%). The activity is therefore moderate (A = 0.62). The particle specific weight was constant (γs = 2.71-2.72 g/cm³).
Shear Strength from Direct Shear Tests

A large number of mechanical tests were performed. The most relevant for analyzing the failure are direct shear tests. Specimens having 50 mm or 60 mm in diameter and thicknesses of 26 mm were tested in shear boxes previously calibrated in order to compensate for deformations not induced on the clay. All the specimens exhibited a brittle behaviour. Peak strength was found for displacements of 0.5 – 1.5 mm (for a range of vertical stresses of 100 kPa – 400 kPa). Beyond peak, the strength initially drops rapidly, then more gradually. This behaviour is illustrated in Figure 9 for one of the specimens tested.

Residual strength was investigated using two types of tests: ring shear tests on remoulded specimens, and direct shear tests on natural discontinuities. Twelve ring shear tests were performed, on samples recovered in the first 20 m of clay, at normal stresses of either 200 kPa or 700 kPa. An average residual friction angle of 13º was measured. No changes with depth were found. The weathered brownish upper clay levels were also tested and no differences in residual friction were observed. Natural joints found on the large clay blocks deposited by the mudflow were tested in the shear box. The discontinuity was aligned with the shearing plane of the box. Several reversing cycles were applied and displacements in excess of 60 mm were achieved. For vertical stresses in the range 200-400 kPa, a friction angle of 11º was found.
The results of all direct shear tests performed on samples cut vertically are summarised in Figure 10. The average peak strength is obtained for a shear displacement of 1 mm. The open symbols in the figure correspond to the strength measured immediately after the peak at essentially the same displacement. The curved strength envelope is a reasonable approximation to the measured values. A straight line is also fitted to the points. The important result is that the sudden loss of strength results in a destruction of the effective cohesion. An accumulation of shear displacements of 6 mm implies a drop of friction angle down to 18°-20°. Finally, the residual friction envelope, associated with further additional displacements, is also plotted in the figure.

![Figure 9: Direct shear test on a specimen of foundation clay from sample M3, borehole S3-1 (Depth: 39.20 – 40.1 m). Normal effective stress: 400 kPa.](image)

![Figure 10: Direct shear strength envelopes of foundation clay](image)
PORE WATER PRESSURE IN THE FOUNDATION CLAY

Fifteen vibrating wire piezometers were installed in boreholes drilled after the failure. Readings started in April-May 1999 and rapid equilibration, in general, followed. Pore water pressures measured in September 1999, are shown in Figures 11 and 12 for Profiles 4 and 1. The most characteristic result was the high pressure measured in the foundation clay. Profile 4 (Figure 11) is located in the displaced embankment and the measured pore water pressures cannot be interpreted in a straightforward manner because they are a consequence of a series of events:

- The initial pore water pressures, before the failure, which were controlled by the history of embankment construction and the process of pore pressure dissipation.
- The increment of pore water pressure due to the rapid loading induced by the displaced embankment.
- The dissipation that had occurred since the failure on April 25th, 1998, presumably under changed boundary conditions.

On April 25th 1998, the embankment moved to its new position raising the load of foundation points such as P1 in Borehole S-4.1 (Figure 11). The point, therefore, experienced an undrained loading followed by a period of dissipation under newly formed boundary conditions. Other points, such as P2 in the same borehole, probably travelled with the moving dam, but were affected by the rapid unloading associated with the sudden decrease of tailings level in the lagoon. This sequence of events makes any prediction exercise somewhat uncertain. Perhaps the major difficulty is the correct interpretation of the drainage boundaries after the development of the sliding surface in the proximity of the piezometers.

Piezometers were installed in the vicinity of the sliding surface with the purpose of deriving the pore water pressure conditions at the time of failure. It is clear from Figure 11 that high pore water pressures, much higher than the values considered in design, were present in the embankment foundation before the failure. Given the similarities in foundation conditions, Profile 1, located in the non-failed portion of the embankment, 400 m North of the breach (Figure 12), may offer a more direct insight into the pore water pressures existing at the time of failure in the slide area. For Profile 1, data were available upstream of the embankment, in the centre of the embankment and at the downstream toe of the embankment. Profiles of water pressure across the embankment can be plotted (Figure 12). This time the interpretation is more straightforward since the only significant change, from the date of the failure, was a partial unloading of the upstream level of tailings and the reduction of hydrostatic pressure in the lagoon.

It is interesting to note, in Figure 12, that the readings in the piezometers P2, located in a shallower position within the clay, detect the boundary water pressure upstream and downstream of the cut-off wall. Water pressures in the alluvial granular soil downstream of the embankment appear to have a low value, close to the original ground surface, similar to that measured in open tube piezometers. Upstream of the cut-off, the water level in the alluvium is given by the water level in the lagoon and this explains the reading in piezometer P2, in Borehole S1-1. It is clear also that strong vertical pressure gradients exist below the embankment in the vicinity of the position of the sliding surface (in the failed embankment) as a result of the consolidation process in very impervious clay.
SIMPLIFIED CONSOLIDATION ANALYSIS OF THE CONSTRUCTION PROCESS

Estimation of Pore Water Pressures at the Time of Failure
The measured pore pressures in Profile 1 provide an excellent opportunity to establish reliable estimates of foundation water pressures at the time of failure. If measurements given in Figure 12 can be reproduced, then the same calculation procedure can be extended to the conditions prevailing before failure in the slipped embankment. A simple consolidation analysis was developed using the following set of criteria:

- Stresses in the subsoil were calculated through available analytical expression for embankment loading on elastic soil. By means of a superposition of incremental loadings, the actual pattern of embankment construction could be followed. The actual sequence of
geometrical changes in the embankment was introduced as a set of instantaneous changes in geometry at given times.

- Every instantaneous application of loading generates a field of pore water pressures, which has been made equal to the increment of total mean stress. This is considered to be a sufficiently good approximation because, in undrained triaxial tests, an average value of Skempton’s coefficient, A=0.33, was found.
- One-dimensional consolidation in the vertical direction towards the upper alluvium and the lower confined aquifer is applied to every increment of loading. Water levels in the upper alluvium change suddenly at the position of the cut-off wall from lagoon level (upstream of the cut-off) to the ground surface (downstream of the cut-off).
- At every instant of time, pore water pressures are calculated as a superposition of all the consolidation processes assumed in the “discretization” of the history of embankment construction.

A comparison of the calculated and measured pore water pressures under the embankment and the lagoon, for Profile 1, is shown in Figure 13. The calculation simulated the construction process, as well as the reduction in tailings level due to erosion immediately after the failure, the change in boundary condition upstream of the cut-off wall, and the subsequent dissipation time prior to the pore water pressure measurements being performed.

The next step in the calculations was the estimation of pore water pressures under the embankment and lagoon for a representative profile of the Southern part of the East embankment. A change in the vertical density of the deposited tailings was considered. The evolution of pore water pressures at three points under the embankment, in a horizontal plane, 14 m below the ground surface, is shown in Figure 14. Also shown in the figure is the estimated cross-profile of pore water pressure immediately before failure for the indicated horizontal plane (which is very close to the position of the actual basal sliding plane). This calculation shows that the pore pressures on the failure plane prior to failure were high, certainly much higher than the values assumed in design.

Figure 13: Calculated and measured distributions of pore water pressures under Profile 1.
Figure 14: Calculated water pressures and stress ratios under the dam and lagoon, at points A, B and C in a representative profile of the failed southern part of the East embankment.

Stress Ratios
Shear stress, $\tau$, and effective normal stresses, $\sigma_n'$, were also calculated, on the basis of the elastic stress distributions and the current pore water pressures, on horizontal planes and, in particular, on the position of the future failure plane. The ratio ($\tau/\sigma_n'$) has been plotted (as a mobilised friction angle) in Figure 14 for the points selected for representing pore pressures. Also indicated is the calculated absolute value of $\tau$ for a few time values. The mobilised friction angle increases rapidly as the downstream toe of the advancing embankment approaches the point under consideration. As the embankment advances further, the mobilised friction angle decreases because of the increasing “confinement” which implies a decrease in shear stress and a parallel increase in normal stresses. As time elapses, the dissipation of pore pressures also results in an increase of the effective normal stresses. In this way, an advancing peak of mobilised friction follows the forward construction of the embankment. This is clearly shown in Figure 15, where the successive profiles of mobilised friction angle along a horizontal plane ($z = 14$ m) are plotted.
Further insight into the stress changes experienced by the foundation clay is gained if the complete distribution of stress ratios on horizontal planes is plotted for different stages of embankment construction. This is shown in Figure 16 for three heights of the embankment (10 m, 19 m and 27 m). A remarkable result is that the stress ratio reaches a peak at a certain depth within the clay. This is a consequence of the variation of pore pressures, controlled by the clay consolidation, and the distribution of shear stresses below the embankment toe. The depth of the maximum $\tau/\sigma'_n$ increases as the height of the embankment increases. For $H = 10$ m, the maximum $\tau/\sigma'_n$ is found for $z = 10$ m. It increases to $z = 12$ m and $z = 17$ m, when $H = 19$ m and $H = 27$ m. This result indicates the position of the sliding plane, which developed later since the peak in mobilised friction generates the maximum damage in clay strength. In fact, the brittle behaviour of the clay, and the strength data already reported, can be now invoked to advance a qualitative interpretation of the failure mechanism. If bedding planes do not exhibit any effective cohesion, a maximum effective friction angle of around 25º may be assumed for them. Then, in view of Figures 14, 15 and 16, the strength on a critically located bedding plane was exceeded for the first time, when the embankment had a height of 8-10 m. This initial critical point was located at a depth of about $z = 10$ m under the downstream toe of the embankment.

This was the situation as early as 1980-1981. Additional stressing in the following years could not be resisted in some areas of the clay. The brittle nature of the clay allowed additional straining in the vicinity of the critical points in a typical progressive failure mechanism. The forward construction of the embankment resulted, therefore, in an increasing extension of the area affected by progressive failure. The line plotted in Figure 16c joins the points of maximum stress ratio. It is viewed as a first approximation of the position of the failed surface under the embankment. Its position is quite close to the position of the future failure plane. The inclination of this line has probably helped to concentrate critical conditions on a specific bedding plane. The line thus defined is an approximation to the failed or damaged surface immediately before failure.

These results should be regarded with caution because the actual distribution of mobilized friction in a horizontal plane located at a depth $z = 14$ m within the clay foundation. Numbers indicate the time elapsed (days) since the start of dam construction (January, 1978).
friction angles cannot possibly be the distribution shown in Figures 15 and 16 because, at any point in the clay, the stress ratio is limited first by its peak strength, and then by the reduced friction angles in subsequent stages of straining. But it is believed that this simplified analysis developed in terms of elastic solutions and consolidation theory provides an interesting interpretation of the damage mechanisms that presumably took place before the actual final failure. It provides also a reasonable explanation for the position of the future failure surface.

Figure 16: Contours of equal mobilized friction on horizontal planes for three phases of construction, in terms of dam height: a) $H = 10$ m; b) $H = 19$ m; c) $H = 27$ m. The line with the arrow in figure c shows the locus of the maximum mobilised angle of shearing resistance.
ANALYSIS OF THE FAILURE

Limit Equilibrium Analysis
If the actual position of the failure surface is analysed, a drained friction angle of the clay, \( \phi' = 17^\circ \), leads to failure conditions (Safety Factor, SF = 1.007). This friction angle, intermediate between the peak friction angle in direct shear tests (24\(^\circ\)) and the residual value (11\(^\circ\)), is an indication of the existence of progressive failure. The minimum safety factor for the embankment is found, however, for critical surfaces other than the quasi-horizontal basal plane. In fact, if circular failure surfaces are assumed, a minimum value SF = 0.72 is found for \( \phi' = 17^\circ \). If the safety factor is fixed at SF = 1, then \( \phi' \) increases to \( \phi' = 21.5^\circ \) for the most critical deep failure circle. This is a further indication that the failure was significantly controlled by progressive failure phenomena and structural conditions as discussed above.

Finite Element Analysis
A large number of elastoplastic coupled flow-deformation analyses were performed, using the 2-D Plaxis code, with the purpose of increasing the understanding of the mechanisms leading to failure. Only some selected results are shown here. The process of dam construction and tailings impoundment was simulated in 11 steps. Each step was, in turn, divided into an undrained application of loading and a subsequent partial dissipation of the pore pressures until the next undrained unloading is applied. A total of 21 stages of calculation are thus defined. A Mohr-Coulomb elastic perfectly plastic model was adopted for all materials.

In a set of analysis a discontinuity with reduced strength characteristics was located at the position of the sliding plane. This involves an interesting issue because, if the reduced parameters chosen reproduce the final conditions, failure is predicted at a much earlier date (in phase 11, out of a maximum of 21). This result is consistent with previous findings, both in the simplified consolidation analysis and in the limit equilibrium calculation. It was necessary to adopt two sets of parameters: a “more resistant” set for the first part of the analysis, and a “weaker set” for subsequent calculations. This is not a procedure to simulate progressive failure, but it provides a strong indication of the need to reduce the available strength of the clay if the actual rupture mechanism is to be approximated. The following strength parameters were adopted for the analysis with a discontinuity plane:

- Plane of discontinuity - \( c' \): varies between 1 kPa and 15 kPa; \( \phi' = 21.5^\circ \)
- Clay above and below the critical plane - \( c' \): 65 kPa; \( \phi' = 24^\circ \) (mass properties)

A drained cohesion, 15 kPa, was assumed to correspond to the initial phases and \( c' = 1 \) kPa was a further reduction of cohesion for the subsequent phases.

This FE analysis also provided data relating to the evolution of pore water pressures in the foundation. A profile of water pressures along the future failure plane is shown in Figure 17. The FE prediction is compared with the result of the elastic-consolidation analysis reported before. The correspondence is very good.

The analysis also confirms that the shear deformations accumulate on the potential failure plane and they extend as the embankment construction advances (Figure 18a). The rupture mechanism, identified by means of the mesh deformation (Figure 18b) is similar to the actual failure mechanism described above. The failure phase was induced by a final reduction of strength parameters. Figure 18b shows the passive wedge developed at the distal end of the
slide and a classic upstream active wedge (the model did not include the vertical jointing existing in the clay unit).

Analysis Post Failure
An analysis of the dam and its foundation just after the onset of failure was also performed (Alonso & Gens 2006b) but the details are outside the scope of the paper. It can be mentioned, however, that the analysis of the dynamics of motion was able to reproduce in a robust manner several features of the failure including the distance travelled by the dam. The analysis provided information on several unknown aspects of the motion. It was found that the motion was rapid and it came to rest 15 seconds after the initiation. The calculated maximum velocity and acceleration (20 km/h and 0.14 g, respectively) provide key insights for a better understanding of the slide and, more importantly, of its consequences.

It has also been shown that the motion stopped because the driving force was reduced, due to the reduction in the level of the volume of liquefied tailings. This reduction is the consequence of the movement of the slide itself, which enlarged the basin being opened upstream of the dam. It is also interesting to realise that the (moderate) increase in resistance, offered by the passive wedge being developed at the distal extreme of the slide, played a very limited role in stopping the motion.

![Figure 17: Distribution of pore water pressures along the failure surface (reference plane). Comparison of FE and simplified elastic-consolidation analysis. Horizontal coordinates start at the upstream toe.](image-url)
Figure 18: (a) Contours of accumulated shear deformations from the origin of calculations (maximum deformation: 22.35%); (b) mesh deformation during the failure process. The figure shows the development of an active state in the tailings, upstream of the dam, and a passive wedge at the downstream toe. The former is not supported by observations but the position of the passive wedge, immediately downstream of the dam is closer to real conditions.

DISCUSSION AND CONCLUDING REMARKS

The Aznalcóllar failure is an unusual case of deep translational sliding involving the entire dam, which displaced a large distance (more than 50 m in the central part) as a rigid body and suffered only minor distortions. It is also interesting to note that the failure did not involve any shearing of the tailings or the rockfill. Both are materials with a high friction angle. Consequently, the stability was controlled by only one material: the highly plastic and brittle Guadalquivir clay. It is believed that the stability of the tailings considerably reduced the consequences of the failure, both in terms of the size of the embankment opening (controlled by the displacement of the embankment), and in terms of the total volume of the spill.
Turning now to the clay foundation, it is clear that the clay had a strong potential for progressive failure given its brittleness. In addition, the downstream construction of the embankment is a process that favours the development of this mechanism as explained above. The large quasi-horizontal displacement of the dam is another interesting feature of this failure, which has to be related to the evolution of driving and resisting forces, once the failure has initiated. The brittleness of the clay and the low residual friction angle indicate that, once the failure has initiated, there is a potential for an accelerated motion due to the progressive loss of clay strength.

The stability analysis performed using finite elements indicates that the measured average peak strength properties of the clay do not lead to the initiation of yield conditions in any point of the clay foundation. It is implied that some strength loss was initially present along the bedding planes. The amount and distribution of the associated reduction of available strength of the bedding planes is not known. Field evidence showing the existence of striated bedding surfaces has been found but the soil investigation performed indicates also that bedding planes were difficult to find. In fact, an engineering interpretation of the borehole records and specimen descriptions would imply a fairly homogeneous and intact massive deposit of clay. Under these conditions, conventional stability calculations predict a rotational failure mode, which was clearly not the case.

The failure was clearly controlled by particular surfaces and this implies that the analysis should concentrate on the available pore water pressures and drained strength properties along the critical planes. It has been argued that the strength had an original reduced strength if compared with the matrix peak strength. The subsequent evolution of clay straining, as the dam was built in a downstream manner and the level of heavy tailings increased, led to a progressive reduction of available strength. The effective cohesive term is lost first as suggested by the interpretation given to the results of drained direct shear strength tests. Thereafter, the failure surface behaves as a frictional contact whose friction coefficient reduces towards an ultimate residual value when the clay structure is oriented in the direction of sliding.

The failure is explained by an average friction angle of the sliding basal plane of 17º-18º (and no cohesion). This value is lower than the average peak friction measured in direct shear tests (24.1º) and corresponds to an average friction intermediate between the peak intact or remoulded friction and the residual friction angle of 11º. As with other sliding failures in brittle clays, a residual factor close to R = 0.5 seems to apply in the case of Aznalcóllar dam.

The reduction of available strength due to progressive failure is, however, far from explaining the failure due to the conservative assumptions adopted in design calculations. Pore water pressures were conventionally taken as given by a steady state flow net, a common practice in dam design. In addition, the water levels measured in open pipes located under the dam in 1996, were taken as an indication of a general phreatic surface located within the surface granular terrace layer. Limit equilibrium calculations using this phreatic level indicated a safety factor against downstream sliding of 1.40 for the dam configuration existing in 1995 (27 m above foundation). The two phreatic levels used in design calculations are compared in Figure 19 with the best estimate of water pressures prevailing on the sliding surface at the time of the failure.

The steady state flow conditions are irrelevant in this case, given the low permeability of the clay foundation. The slow dissipation of pore water pressures is also a consequence of the
considerable homogeneity of the clay, in terms of permeability. No pervious seams or structural discontinuities helped to dissipate excess pore water pressures. The joints and bedding surfaces are not conductive features. Remarkably, the consolidation coefficient derived from field observations of pore water pressure gradients is very similar in this case to the coefficient of consolidation derived from the back analysis of the consolidation behaviour of small specimens. The average degree of dissipation of pore water pressures after 20 yrs of continuous increase of the dam height is no more than 15%.

![Diagram](image)

**Figure 19:** Phreatic surface used in design calculations and best estimate of pore water pressure existing on the sliding plane at the time of the failure

A final point relates to the slopes of the rockfill dam. They were designed to be 27.8° and 29.9° (upstream and downstream respectively). The downstream slope was increased to 39° in around 1985 and it was maintained constant thereafter. The increase in slope reduced the safety of the dam and extended the damage associated with progressive failure. It has been shown that the geometry of the advancing dam and the parallel partial dissipation of pore water pressures generate a critical position for the maximum of shear stress ratios. These critical points, combined with the downstream construction method used, seem to define the position of the rupture surface. A (hypothetical) new design of the dam would certainly require a significant reduction of the slope of the downstream slope, in order to maintain the induced shear stress ratios below the existing strength on the bedding surfaces.

**REFERENCES**


ACKNOWLEDGEMENTS
The authors are grateful to their colleagues J. Alcoverro, A. Lloret, C. López, J. Moya and E. Romero for their assistance during the investigation on the Azanalcóllar dam failure.
REFLECTIONS ON FORENSIC INVESTIGATIONS TO DETERMINE THE NATURE, RISK, AND CAUSES OF THE 1979 ABBOTSFORD LANDSLIDE, DUNEDIN, NEW ZEALAND

Graham T. Hancox
Hazards Group, Institute of Geological and Nuclear Sciences
Lower Hutt, New Zealand

Abstract: The Abbotsford Landslide of 8 August 1979 occurred in an urban area of Dunedin, New Zealand, causing damage to houses and urban infrastructure. Rapid failure occurred after weeks of preliminary movements, resulting in the formation of a c. 5 million m\(^3\) block slide. It caused the loss of 69 houses, with an overall cost of about 13 million (NZ) dollars. Prior to the failure, geological field mapping, survey monitoring and drilling was undertaken to establish the nature of the landslide and rate of movement. Inclinometers in drill holes showed the depth of movement, but insufficient geotechnical and piezometric information, was collected to allow the landslide to be analysed, remedial actions taken, or evacuation of residents before the final movement. Extensive ‘forensic’ geotechnical investigations were carried out after the failure to determine the cause of the landslide and long-term stability of the area. Those investigations included cored drill holes, inclinometers, piezometers, test pits and calweld shafts, and laboratory soil testing (grading, bulk density, consolidation, permeability, Atterberg limits, shear strength, and mineralogy). This paper describes the development of the Abbotsford Landslide, presents a summary of the pre and post-slide investigations undertaken to evaluate it, and examines the scope and adequacy of the investigations for assessing and responding to the risk posed by the landslide, and subsequently for determining its cause.

INTRODUCTION
Rapid movement of the Abbotsford Landslide in the Abbotsford suburb of southwest Dunedin, New Zealand (Figure 1) occurred on 8 August 1979 following several weeks of slow movements. No one was injured or killed by the landslide, but because it occurred in an urban area and caused damage to many houses, it is New Zealand’s most famous landslide in the last 50 years.

The Abbotsford Landslide became the subject of a Government Commission of Inquiry (CoI) in 1980 to determine the cause of the disaster, the role of human activities in its development, and the adequacy of measures taken before, during and after the event. Twenty eight years after it occurred, this landslide still offers valuable lessons about the causes of such events, both natural and man-made, measures to avoid or mitigate the effects of such events in the future, and also the scope and role of ‘forensic’ geotechnical investigations in assessing landslides and associated hazards.

The author was involved in investigations after the landslide to determine its cause, and was co-author of several reports (e.g. Bishop et al. 1979; Hancox et al. 1980; Salt et al. 1980) prepared by DSIR New Zealand Geological Survey (now part of the Institute of Geological and Nuclear Sciences) for the 1980 Commission of Inquiry into the disaster. A recent paper (Hancox 2008) provides a historical case history summary of the geotechnical aspects of the landslide, including discussion of its development history, the underlying preconditions that contributed to the failure, and likely triggering factors.
Figure 1: Location of the Abbotsford Landslide

Drawing on information presented in the earlier reports and recent case history (Hancox 2008), this paper describes the setting and nature of the Abbotsford Landslide, and presents a summary of the pre and post-slide forensic geotechnical investigations carried out to determine the nature, risk, and causes of the Abbotsford Landslide. The scope of these investigations are then discussed and examined in relation to their effectiveness in assessing and managing the hazard and risk presented by the landslide as it developed, and what factor or factors triggered the movements and sudden collapse in August 1979.

LOCATION AND GEOLOGICAL SETTING OF THE LANDSLIDE

The Abbotsford Landslide is located in a residential suburb in southwest Dunedin (Figure 1). The terrain is gently hilly with moderate relief. The landslide lies on the eastern side of a gently sloping spur on the northern side of Kaikorai valley, and is bounded to the east by Miller Creek. The landslide area is underlain by weak, Tertiary age sedimentary rocks (Figure 2).

The spur on which the landslide is located rises steeply (20–25°) from the Kaikorai valley floor for about 100 m (elevation) but flattens off near Abbots Hill Road at the top of the slope. The eastern side of the spur had earlier been modified by a sand quarry (Harrison’s Pit) and the adjacent Sun Club Slide, an inactive old landslide at the northern end of the quarry (Figure 2).

Geology of the Landslide Area

As shown in Figure 2, the sequence of Tertiary sedimentary rocks underlying Abbotsford is about 600 m thick with a uniform regional dip of 6–9° to the southeast, and overlies Jurassic age schist basement rock (Benson 1967; Hancox et al. 1980; McKellar 1990). The rocks directly underlying the landslide include the 250 m thick Abbotsford Formation – a weak green-grey mudstone, with sand lenses and thin montmorillonitic (smectite) clay layers. This formation is overlain by up to 100 m of Green Island Sand – a weak, non-cohesive, pale yellow brown clayey to silty sand.

Most of the Tertiary rocks in the Abbotsford area are mantled by Quaternary age surficial deposits of dense, clayey bouldery colluvium c. 5–10 m thick, which is overlain in most
places by up to 7 m of stiff, clayey loess (not shown on Figure 2). These deposits result from solifluction and aeolian processes during the Last Glaciation about 14,000–20,000 years ago.

Figure 2: (a) Geological map of the Abbotsford district showing locations of the Abbotsford Landslide and other large landslides in the surrounding area; (b) cross section A–B shows the topography and geological structure of the area in relation to the Abbotsford Landslide.

Other Landslides in the Abbotsford Area
Deep-seated slides and flows are common in Tertiary rocks in the Dunedin area, particularly in the Abbotsford Formation and Burnside Mudstone (Benson 1940, 1946; Hancox et al. 1980; McKellar 1990). Prior to 1979, several large landslides and areas of soil creep had been identified in the Abbotsford area (Figure 2). The West Abbotsford Landslide is a large prehistoric (about 9,000 years old) earth flow in Abbotsford mudstone. The toe of that slide was reactivated in 1968 by a cut for the State Highway 1 motorway, causing damage to houses. The Sun Club Slide is an inactive (relict) prehistoric landslide (c.10,000 years B.P.) in Green Island Sand on the western side of Miller Creek (Hancox et al. 1980). This landslide was well known before the Abbotsford Landslide occurred (Benson 1946; Gordon 1960), and clearly demonstrated the potential instability of the dip slope underlying Abbotsford.

Harrison’s Pit Sand Quarry
Between 1934 and 1969, Green Island Sand was excavated from Harrison’s Pit sand quarry at the toe of the southern half of the slope that failed in 1979 (Figures 2 and 3).
Approximately 330,000 m$^3$ of sand was excavated from the base of the slope, most of which was removed between 1964 and 1969 for construction of a nearby motorway. Much of the sand was used to stabilise (buttress) the toe of the West Abbotsford Landslide, which was reactivated in 1968 (Figure 2). The quarry excavation was about 90 m wide, with four batters constructed and an overall slope of about 25°, similar to the original slope (Figure 3). The sand quarry was closed in 1969, ten years before the final catastrophic movement in August 1979.

**DEVELOPMENT AND INVESTIGATION OF THE LANDSLIDE**

**Early Landslide Movements**
The Abbotsford Landslide did not occur without warning. Minor cracking damage was reported to one house 60 m south of the sand pit (Figure 3) as early as 1968–72, which coincided with removal of a large volume of sand from the pit. That house may have been damaged by initial creep movements of the landslide (CoI 1980), but there was not general agreement on this point. Damage to the house between 1968 and 1972 was thought by the authorities to be unrelated to the sand pit excavations or any major land movement. However, that house was one of the first to be seriously affected by early movement of the landslide in 1978, when ground movements in the wider area first became obvious. Between September 1978 and June 1979, several water main breakages and damage to houses in Christie and Mitchell streets (Figure 3) were caused by growth of ground cracks, which became more apparent as they spread northwards across adjacent farmland. These cracks eventually defined the head scarp and graben of a developing large landslide (Figures 4 and 5).
Pre-slide Geotechnical Investigations

Geotechnical investigations to determine the nature of the developing landslide at Abbotsford and the depth of movement began in June 1979. The pre-slide investigations included:

(a) Establishment and monitoring of survey marks and two survey lines.
(b) Recording of damage to houses, services (water pipes, overhead wires).
(c) Mapping of ground deformation features (tension cracks, shear cracks, compression ridges, settlements and subsidence).
(d) Drilling of 6 small-diameter drill holes (N-size, triconed and cored), with tube inclinometers installed in 4 drill holes.
(e) Several shallow (4-6 m deep) test pits and push-tube samples.

Details of the pre-slide drill holes, inclinometers, and recorded water levels are summarised in Table 1. Figure 5 shows the locations of damage to houses, breaks in water mains, ground deformation features, survey lines, drill holes and test pits before the final movement on 8 August 1979. Plots of slide movement rates and crack widening determined from ground surveys are shown in Figure 6.

Although there was evidence of slope creep in the Abbotsford Landslide area between 1958 and 1978, the first unequivocal proof of ground movements associated with the 8 August failure were confirmed on 30 May 1979 when a pulled-apart water main joint was discovered in Mitchell Street (Col 1980; Hancox et al. 1980). Fresh ground cracks and movements in several adjacent houses were also found, so a full-scale investigation was launched on 1 June to determine the extent and cause of this damage, and began with installation of ground survey marks around the affected area.
Figure 5: Map of the Abbotsford Landslide area showing the pre-slide topography, ground cracking and deformation features, houses damaged by the initial slide movements (1, 2, 3), and investigation drill holes, test pits, and survey lines (after Hancox et al. 1980).
### Table 1: Details of pre-slide drill holes, inclinometers and groundwater levels

<table>
<thead>
<tr>
<th>Drill Hole Number</th>
<th>Date Drilled (m)</th>
<th>Depth (m)</th>
<th>Groundwater Depth (m)</th>
<th>Inclinometer Data Installed</th>
<th>Failed</th>
<th>Failure position and time</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>22/6/79–29/6/79</td>
<td>15.9</td>
<td>9.3</td>
<td>31/7/79 [4th]*</td>
<td>1/8/79</td>
<td>Depth 17.6 m after 10 hrs* (0.9 m below GI Sand / Abbotsford mudstone contact).</td>
</tr>
<tr>
<td>M2</td>
<td>30/6/79–9/7/79</td>
<td>21.2</td>
<td>0.8</td>
<td>10/7/79 [1st]*</td>
<td>12/7/79</td>
<td>Depth 12.8 m after 2 days* (0.4 m below GI Sand / Abbotsford mudstone contact).</td>
</tr>
<tr>
<td>M3</td>
<td>10/7/79–26/7/79</td>
<td>36.8</td>
<td>14.2</td>
<td>27/7/79 [3rd]*</td>
<td>27/7/79</td>
<td>Depth 29.5 m after 17 hrs* (0.7 m above GI Sand / Abbotsford mudstone contact).</td>
</tr>
<tr>
<td>M4</td>
<td>21/7/79–26/7/79</td>
<td>18</td>
<td>2.2</td>
<td>26/7/79 [2nd]*</td>
<td>27/7/79</td>
<td>Depth 13 m after 26 hrs* (0.85 m below GI Sand / Abbotsford mudstone contact).</td>
</tr>
<tr>
<td>M5</td>
<td>c. 2/7/79–2/8/79</td>
<td>29</td>
<td>5.3</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
</tr>
<tr>
<td>M6</td>
<td>4/8/79–8/8/79</td>
<td>17.5</td>
<td>2.3</td>
<td>NA</td>
<td>NA</td>
<td>-</td>
</tr>
</tbody>
</table>

**Notes:**
Drill hole locations are shown on Figure 5. Groundwater levels are average water depths in drill holes left overnight. Piezometer tubes installed in drillholes M2, 3, 4, and 6 became blocked or sheared off by ground movements (no reliable readings). * Note that the time to inclinometer failure after installation becomes progressively shorter, showing that the landslide was accelerating (as shown in Figure 6).

**Figure 6:** Graphs showing movements of the Abbotsford Landslide from 18 June to 8 August 1979. Ground movement rates of survey points and lines in Mitchell and Edward streets, are shown in relation to crack development, damage to water mains, and rainfall (mm/day)
Over the next 11 days, these survey marks showed movement rates of 3–23 mm/day. Survey monitoring lines were then established across the widening area of ground cracks in Mitchell Street and Edward Street. The ground surface deformation features suggested that deep-seated landslide movement was occurring. To determine the extent and depth of the movements, drilling investigations began in Mitchell Street on 22 June 1979 (drill hole M1). Over the next six weeks, 6 drill holes and several test pits provided information on the thickness of the landslide, the depth of the slide plane, soil properties, groundwater levels, and near-surface geology (Figure 5). Tube inclinometers were installed in four drill holes (M1–M4), but these were all sheared off by slide movement 1–2 days after installation, at least a week before the final movement. The time to inclinometer failure after installation also became progressively shorter, indicating that the landslide was accelerating (Table 1).

Results of Drilling Investigations and Landslide Monitoring
The mapping of ground deformation features, damage to houses and services, drillholes and inclinometers showed that the landslide was large and deep-seated, with the failure plane located near the top of the Abbotsford Mudstone; in places 30 m below the ground surface (Table 1). Repeat surveys of monitoring marks and lines during June and July 1979 showed increased ground movements, with rates of 10–20 mm/day from June to mid July. The movement rate increased to about 100 mm/day at the end of July after 55 mm rain over 7 days, clearly showing that the ground movements were responding to rainfall (Figure 6).

Over June and July 1979, the ground cracks in Mitchell Street began to define a graben. As movement accelerated after 26 July, the graben cracks propagated quickly northwards across farmland behind the Sun Club Slide (Figures 4 and 5). Ground deformation was also recorded at the bottom of the slope in Harrison’s Pit, and at the toe of the Sun Club Slide, where 30 m long cracks and turf compression rolls developed. By 4 August, the graben cracks had extended more than 100 m north of the Sun Club Slide. On the afternoon of 8 August, shear cracks were found extending from the northern graben crack down to Miller Creek, defining the full extent of the area affected by ground movements (c.20 ha) and clearly outlining the considerable size of the slowly developing landslide (Figure 5).

Assessment of Landslide Risk and Response
In late June 1979, a geotechnical committee of local engineers and geologists was formed to investigate the landslide. Throughout July, the focus of the committee was mainly on defining the depth and mechanism of the landslide, measuring groundwater levels, and monitoring rates of ground movement. Because of the size of the affected area and accelerating rate of movements, remedial measures to stabilise the area were judged to be impractical (CoI 1980). Groundwater levels in the affected area were high (Table 1), so deep drainage was considered to be a possible solution (especially in the permeable Green Island Sand), but was never initiated, mainly because of time constraints rather than cost.

There was growing concern among engineers and geologists on the committee about the increasing rates of ground movement, so after the last inclinometer failed at the beginning of August, the emphasis shifted towards using movement rates to assess the risk posed by the landslide if a sudden, rapid failure and break up of the slope occurred. On 5 August, when movements had accelerated to 150–200 mm/day (Figure 4), the geotechnical committee predicted that movements of 450 mm/day were likely by 12 August (CoI 1980). Because of this possibility, and increasing concern and uncertainty about when and how slope collapse might occur, a state of local civil defence emergency was declared by the Mayor of Green Island Borough (GIB) at 8 a.m. on Monday 6 August 1979. Under provisions provided by the

436
state of emergency, residents in the area of ground movements (Figure 5) were advised by the GIB Council of the increasing landslide risk, and that while there was no immediate cause for concern, they should make arrangements to vacate their homes (CoI 1980).

At mid day on 6 August, a progressive evacuation plan was adopted by the civil defence authorities. This plan required evacuation of the houses most at risk first (those in Christie, Mitchell, Edward, and Charles streets), and would proceed until all houses at risk were vacated by 12 August. That date was chosen to correspond to a predicated movement rate of 450 mm/day and 4 m total displacement, beyond which services (water, power) to houses on the sliding mass could not be maintained. The evacuation proceeded slowly over the next two days. Despite concern about ongoing slide movements, it was not appreciated over those days that the slide was in fact moving faster than had been predicted. Survey data compiled several days later (Figure 6) indicated movement of about 430 mm/day on 6 August, increasing to 670 mm/day on 7 August (Hancox et al. 1980). Although the precise rates of movement were not known at that time, anecdotal reports of steady movement of pegs and widening of cracks, suggested that the slip situation was worsening sooner than expected (CoI 1980).

Because there was still uncertainty about how rapidly the slide would develop, some residents were reluctant to leave their homes. However, by the evening 7 August, most of the affected area had been evacuated, and by 8.30 p.m. on 8 August, only about 6 houses were still occupied. When the final movement of the landslide began at 9.07 p.m. that evening, 17 people were carried down the slope in their homes, and marooned on the slide mass when access was cut off by a gaping chasm, into which houses had tumbled and broken up. All of the stranded people were rescued by 11 p.m. Because of concerns about further slope collapses, areas to the west and south of the landslide were evacuated by 1 a.m. on 9 August. A total of 640 people were displaced from their homes as a result of the landslide (CoI 1980).

Final Landslide Movement
The final rapid movement of the landslide lasted about 50 minutes. A large block of land containing 28 houses and 17 people slid southeast towards Miller Creek. The sliding mass left behind a large chasm (graben) up to 30 m deep, in which 15 houses were destroyed (see Figures 7 to 9).

The landslide moved 50 m downslope towards Miller Creek over about 30 minutes, forming a graben 70–150 m wide and 15–30 m deep at the slide head. Much of the sliding block remained intact. Miller Creek was temporarily blocked by landslide debris, which included remnants of the relict Sun Club Slide (Figure 8). A trench was quickly dug to drain the ponded water and prevent a dam-break flood. Because the movement was relatively slow, and people had been evacuated from areas of greatest damage (in the graben and at the toe), no lives were lost and no one was injured by the slope failure. The landslide resulted in 69 houses being either destroyed or rendered inaccessible by the graben (Figures 8 and 9). Many houses on the sliding block survived the landslide and were later transported away from the area to new sites elsewhere. The Commission of Inquiry found that the declaration of the state of emergency by the Mayor of Green Island was a timely and courageous decision, which was made despite opposition due to liability concerns, and the council could not be faulted for the part it played as the landslide movements increased (CoI 1980).
Figure 7: Annotated aerial photo of the Abbotsford Landslide on 9 August 1979, the morning after the failure, showing its effects and main geomorphic features (Photo by courtesy of Aeropix, Dunedin).

Description of the Landslide
Investigations after the movement on 8 August showed that the Abbotsford Landslide was a deep-seated block slide (as defined by Cruden & Varnes 1996). The landslide affected 18 ha, and had an estimated volume of about 5 million m$^3$ (Hancox et al. 1980). The bulk of the landslide mass slid 50 m down a 7° dip-slope in 30 minutes (average speed 1.7 m/min). A graben formed at the slide head, with compressional ridges and sand flows at the toe. Figure 10 shows these and other geomorphic features of the landslide. The geology and subsurface details of the landslide are shown by two cross sections (Figure 11).

The landslide mass contained well defined geomorphic features and deformation zones. The extensive and chaotic graben at the slide head was the most obvious feature. Most of the slide mass was undeformed, but was separated by medial shear cracks into two slide blocks. The southern Charles Street Block formed above the sand pit and over-ran the western side of Miller Park. The northern Sun Club Block developed below the widest part of the graben. This block contained the largely intact Sun Club Slide, where its toe dammed Miller Creek (Figure 8). Both blocks had wide zones of compression ridges and long over-thrust shears at the toe (Figures 10, 11, 12).

There were also complex lateral zones of shear deformation at the northern and southern ends of the landslide mass. The toe of the landslide comprised mainly Green Island Sand (Figure 11). This material was the source for sand and debris flows seen at the toe after movement of the main slide blocks had ceased (CoI 1980). Sand at the slide toe flowed (rather than liquefied) as groundwater drained from the slope after failure. Investigation drill holes and shafts completed before and after the failure showed the slide plane was located in a clay layer at the top of the Abbotsford Formation (Figure 11).
Figure 8: Aerial view of the Abbotsford Landslide showing the graben (g), earthworks (ew) to regrade the head scarp (hs) to protect nearby houses, the old sand pit (sp), the Sun Club Slide (sc), and water ponded (p) in Miller Creek by slide debris. Many houses (h) on the slide mass were largely undamaged (Photo by courtesy of G Hancox).

Figure 9: The scene on the morning of 9 August showing the chaotic slumping and some of the houses destroyed in the graben when it opened. The tilted and damaged house (h) moved about 50 m southeast and down about 12 m by the collapse (Photo by courtesy of Don Bird, EQC).
Figure 10: Map of the Abbotsford Landslide showing the post-slide topography, geomorphic features and zones of the landslide, and investigation drill holes and test pits (after Hancox et al. 1980).
Figure 11: Cross sections of the Abbotsford Landslide showing the pre and post-slide topography, landslide features, and some investigation drill holes, test pits and shafts (after Hancox et al. 1980).

Figure 12: Photos showing features in the Sun Club (SC) at the toe of the landslide (Sun Club Block). Compressional deformation formed ridges (cr) destroyed the tennis court (TC, a and b), tilted large pine trees (c), and deformed the swimming pool (SP) and moved it 38 m to the southeast. A low-angle shear plane (tpl) which has under-thrust the tennis court from right to left (b), defines the western boundary (uphill) of the zone of compression. (Photos (a) and (b) by courtesy of G Hancox; (c) D Coombs).
Post-slide Geotechnical Investigations
Geotechnical investigations of the Abbotsford Landslide continued after the final movement on 8 August 1979. The investigations began on 11 August, primarily to assess the stability of the landslide area and the risk of further slope movements. When the New Zealand Government’s Royal Commission of Inquiry into the Abbotsford Landslip Disaster was announced on 20 August, the scope of the investigations widened to include establishing the cause of the landslide. The post-slide investigations and activities included:

(a) Drilling of 18 small-diameter (N-size) fully cored drill holes, 11-59 m deep (M11–28, see Figures 10 and 11).
(b) Inclinometers in 8 drill holes (M11–17, 26), with standpipe groundwater monitoring.
(c) Sealed piezometers in 6 drill holes (M12–22, 27, 28).
(d) Six back-hoe test pits, 3–6 m deep (TP 12–15, 22, 23).
(e) Seventeen Calweld shafts (900 mm), 10–30 m deep (TP 16, 18–21, 24–35).
(f) Soil testing (Atterberg limits, consolidation, densities, grading, mineralogy, permeability, and triaxial (sand) and shear box (clay layers) strength).
(g) Geomorphic and geological mapping (see Figures 10 and 11).
(h) Examination of the geological history and historical records of landslides and ground movements in the area, rainfall and earthquake data, pre and post-slide groundwater conditions, natural erosion, and the activities of man (removal of vegetation, sand pit excavation, leakage of water pipes, urbanization of area).
(i) Limit equilibrium slope stability analyses.
(j) Evaluation of conditions that contributed to the cause of the landslide, and the main factor(s) that triggered movements leading to the final slope collapse.

The information obtained from these investigations was shared by all organisations involved, who prepared their own submissions on the cause of the landslide. The investigation results and interpretations by DSIR were presented in several reports (Bishop et al. 1979; Hancox et al. 1980; Miller & Turnbull 1979; Northey & Barker 1980; Salt et al. 1980), which were submitted to the Commission of Inquiry and taken into account in their report on the disaster (CoI 1980).

THE CAUSE OF THE LANDSLIDE

Information from Geotechnical Investigations
During the pre-slide investigations, inclinometer shear points in four drill holes showed that the slide plane was located 10–30 m below the ground surface, close to the top of the Abbotsford mudstone (Table 1). This was confirmed after the 8 August movement when the failure surface was located in a 25 mm thick clay layer, about 1 m below the interface between the Green Island Sand and the Abbotsford Formation (in TP 34, Figures 10 and 11). The clay layer, which showed slickensides indicating past movements, contained up to 25% montmorillonite clay (Northey & Barker 1980) and had very low residual shear strength (ϕ, internal friction angle) ranging from 5–10° (Millar & Turnbull 1979). Montmorillonite is a highly active swelling clay mineral that is well known for causing slope instability problems in Dunedin and other parts of New Zealand (Crozier et al. 1992; Stout 1971). Because the clay layers were on the slide plane, they were foremost among a number of factors examined to determine the cause of the landslide.
The 8 August collapse proved to be the final movement of the Abbotsford Landslide. A number of northwest-trending slump scarps up to 2 m high developed across the end of Charles Street a week after that movement (Figure 10), but no further ground displacements were detected in the area from August 1979 to March 1980 (Hancox et al. 1980), and none have occurred subsequently. Following the August 8 movement, the landslide area was found to be stable with a long-term factor of safety of 1.10, without the need for any stabilization measures (Salt et al. 1980). The built-up areas adjacent to the landslide area south and west of Christie Street were also judged to be stable (FS=2.2). After substantial earthworks were carried out to regrade the headscarp and fill in the graben, the area was re-zoned as a reserve in which all future building and development has been prohibited.

Factors that Contributed to the Cause of the Landslide

A range of natural and human factors were examined by the organisations involved in the investigations (DSIR, Ministry of Works and Development, Otago University, local authorities) to determine the cause of the landslide. The principal factors included: unfavorable geology and topography, increased groundwater levels (due to higher rainfall and leakage of water pipes), quarrying of sand at the toe of the slope, removal of the natural vegetation, urban development of the area, and seismic activity (CoI 1980; Hancox et al. 1980; Salt et al. 1980). These factors and their effects are summarised in Table 2.

Several submissions to the Commission of Inquiry (e.g. Coombs et al. 1980; Hancox et al. 1980; Salt et al. 1980) concluded that the presence of thin clay layers in the 7–10° dip slope was of fundamental importance in the development of the landslide. However, other factors also contributed to the failure, notably the sand quarry at the toe of the slope, high groundwater levels from greater than normal rainfall over past decade, and leakage from broken pipes within and immediately above the landslide area.

Limit equilibrium analysis showed that excavation of 300,000 m$^3$ of sand from the base of the slope reduced stability of the overall slope by about 1%, but was insufficient to trigger the landslide (Salt et al. 1980). The early (1968–1972) damage to a house 60 m south of the sand pit (3, Figure 5) may have been caused by initial movements of the landslide (CoI 1980). Although the sand excavation did not cause the slope to fail immediately, it did reduce its stability, and when combined with other factors it probably advanced the onset and timing of the slope failure process. From the evidence presented, the CoI (1980) found that the excavation of Harrison’s Pit was a causative factor in the landslide.

There was little reliable information about the history of groundwater levels in the landslide area before the slope failed; therefore the CoI considered it was not possible to determine the proportionate effect of the sand pit excavation. A similar (1%) reduction in stability would have resulted from a uniform 0.3 m rise in groundwater level over the slide area. A rise in the long-term groundwater levels in the area was considered likely because of increased rainfall over the past decade, and a leaking water main (a segmented 750 mm concrete pipe with rubber ring joints) about 100 m upslope of the landslide (Figures 10 and 13). Although the exact volume of water that leaked from the water main is unknown, the Dunedin City Council estimated that, between 1976 and 1979, it was about 5 million litres per year. This suggested an average leakage rate of 6–9 litres per minute, of which 4–5 litres per minute was likely to have infiltrated into the ground in the landslide area (CoI 1980).
Table 2: Summary of factors that may have contributed to the Abbotsford Landslide

### MOST SIGNIFICANT

1. **Unfavorable Geology and Topography:**
   - (a) Landslide formed on a dip slope - Tertiary sediments dip c. 7–10º towards Miller Creek (Figure 2).
   - (b) Very weak clay layers at near residual strength ($\phi'$ 5–10º) aligned with bedding at top of Abbotsford Formation.
   - (c) Progressive failure over several thousand years reduced strength of clay layers to close to residual strength. This process probably began after downcutting by Miller Creek more than 20,000 years ago.
   - (d) The prehistoric Sun Club Slide (c. 10,000 years BP) illustrates the inherent instability of the slope on the west side of Miller Creek. Slope stability was further reduced by stream erosion at the toe of the old slide.

2. **Increased pore water pressure**
   Limit equilibrium analyses showed that the slope that failed was sensitive to changes in the water table. A uniform rise of 1 m in groundwater level in the area was found to reduce slope stability by 2-3%. A long-term rise in the water table in the landslide area is likely to have occurred because of two important factors:
   - (a) Increased rainfall over the last 10 years prior to the failure (following a 20-year drier period).
   - (b) Leakage from the Dunedin City Council (DCC) 750 mm water main at the top of the slope (Figure 10).

3. **Quarrying of sand at the toe of the slope**
   - (a) Excavation of about 300,000 m³ (6% of the slide mass) from Harrison’s Pit between 1964 and 1969 reduced the stability of the overall slope by c. 1–2 % (locally the reduction would have been higher). This was not sufficient to initiate immediate failure of the slope.
   - (b) A 1% reduction in the stability of the overall slope would also have resulted from a uniform 0.3 m rise in groundwater levels, which equates to a 3% decrease per meter rise in groundwater level.

### LESS SIGNIFICANT

4. **Removal of natural vegetation**
   Removal of native bush and scrub cover in the area more than 100 years ago (before 1894), possibly had a minor effect on stability due to slightly increased surface runoff, and hence a rise in the water table. The lack of data about previous vegetation and runoff characteristics made it impossible to quantify these factors.

5. **Urban development (excluding DCC water main)**
   Building of houses, roads, and services resulted in a very small increased surcharge (weight) on the slope over 15% of the slope area. Increased interception of rain (on houses, roads) possibly offset deficiencies in storm water reticulation. Breaks in water mains, and pipe for East Christie St stream (1978 and 1979) resulted from early slide movement. Most were detected and repaired. Vibrations from motorway too small to have affected slide area.

6. **Seismic activity**
   Not relevant. No significant earthquakes (MM 5 or greater) in the area in the 3 years prior to the slide. The strongest historical earthquake (M 5, MM VII in April 1974) is not known to have affected the slope. The effects of prehistoric earthquakes are unknown.
Based on information presented to the CoI, the average annual infiltration rate of rainfall into the landslide area was considered to be about 10 litres per minute. The water leaking from the water main since 1976 was roughly equivalent to a 40–50% increase in rainfall over that period. It is possible, therefore, that the combined effects of greater than average rainfall over the past decade, together with water from the leaking water main, raised the groundwater levels in the landslide area, and so advanced (triggered) the onset of slope failure.

Insufficient reliable data on groundwater levels were obtained during the 1979–80 investigations to precisely determine the effects of leakage from water pipes on the landslide area. Pre-slide drilling reports indicating ‘sounds of water running underground’, and the extensive sand flows at the toe of the slide (Figure 10), suggested that groundwater levels in the landslide area prior to the final movement were very high. The CoI found that, although the exact amount of water reaching the slide area from the leaking water main was unknown, it was probably sufficient for its effect on the slide mass to be significant (CoI 1980).

Findings on the Cause of the Landslide
The main findings of the DSIR on the cause of the Abbotsford Landslide (Salt et al. 1980) may be summarised as follows:

1. The underlying geology and topography of the area created the basic pre-conditions for instability of the dip slope formed in weak sedimentary rocks. The relict Sun Club Slide is evidence of past landsliding adjacent to where the landslide developed.
2. The block slide resulted from translational failure on thin and extremely weak (shear strength $c' = 0–20$ kPa, $\phi' = 5°–10°$), low-angle clay layers located near the top of the...
Abbotsford mudstone, and dipping downslope 7° to the southeast. Progressive failure on the clay layers to near residual strength began several thousand years ago with the erosion of Miller Creek, and was important in the development of the landslide.

(3) Limit equilibrium analysis indicated that the sand excavation at the base of the slope reduced stability of the overall slope by about 1%, approximately equivalent to a uniform 0.3 m rise in groundwater levels (or 2–3% decrease in stability per metre rise).

(4) The failure on 8 August occurred after a decade of higher than average rainfall. This is likely to have decreased stability of the slope, but how much groundwater levels rose as a result of the increased rainfall could not be determined.

(5) There was evidence of substantial leakage from the DCC water main at the top of the slope, possibly equivalent to a 40–50% increase in rainfall over the three years prior to landslide. This could have had significant effect on slope stability, but how far groundwater levels rose due to leakage from the water main was unknown.

(6) Other effects such as the removal of vegetation, urban development, and seismicity were generally thought to be insignificant. The lack of any significant earthquakes in the Dunedin area during the three months prior to the movement on 8 August indicated that the landslide was not triggered by an earthquake.

DISCUSSION

Assessment of Landslide Risk and Response

Investigations undertaken before the final movement of the Abbotsford Landslide on 8 August 1979 accurately defined the rates and depths of the developing ground movements, and the extent of the areas likely to be affected by a sudden collapse of the slope. Survey monitoring across the developing ground cracks showed that the rates of movement were increasing exponentially the week before the final movement, but it was difficult to predict the exact acceleration of the landslide, or how and when the slope would break up. The movement rate predicted on 5 August (450 mm/day and 4 m total by 12 August) proved to be lower than what actually occurred, with much faster movements (430–670 mm/day) recorded by 6–7 August. However, the high risk presented by the landslide was appreciated, as was shown when the state of emergency and evacuation plans were announced on 6 August. When the movement occurred on 8 August, much earlier than anticipated, 17 people remained on the sliding mass.

Although those people were unharmed, their survival was more a matter of chance rather than management. The author recalls that in 1979 it was a widely held view that the people on the landslide should have been evacuated earlier. Unfortunately, over the two days prior to the final movement on 8 August, the survey monitoring data were not plotted up and analysed quickly enough for the rapid increase in movements on 6–7 July to be recognised. This meant that the area was not completely evacuated before the landslide occurred. In spite of anecdotal reports from some residents of increased ground movements over that critical pre-slide period, those reports were not taken seriously and acted on by the authorities.

The Commission of Inquiry found that the local council could not be faulted for the part it played as the landslide developed, and that the declaration of the state of emergency was a timely and responsible action. However, the CoI considered that the consequences of rapid slope collapse would have justified the authorities (civil defence, police) requiring complete evacuation of the landslide area earlier than what actually occurred. Despite the possibility of an over-cautions prediction about how the landslide might develop, the Commission found
that in such cases human safety should be the overriding consideration, and that decisions should be made assuming the “worst possible” rather than the “most probable” eventuality (CoI 1980). That approach to managing landslide hazards and risk remains equally valid today.

**Effectiveness of the Investigations**

The geotechnical investigations established the depth and rates of movement of the developing landslide, and the location of the slide plane within very weak bedding plane clay layers. However, the slide mechanism and (triggering) cause of the landslide could not be conclusively determined because groundwater levels in the area were not accurately defined before the 8 August movement. This critical information was not obtained mainly because there was insufficient time to properly install and prove the standpipes and piezometers or because they were sheared-off or blocked by ground movement. There were also difficulties because there was limited practical experience in investigating a large moving landslide, and the lack of equipment to measure groundwater levels in drillholes that were continuously being deformed and blocked by ground movements.

With new landslide monitoring techniques and modern geotechnical equipment available for collecting piezometric data, better results are now expected. For example, much improved equipment is currently being used by GNS at Taihape in the central North Island to investigate a very large (45ha), deep-seated, creeping landslide with over 200 homes and a school. A laser survey network and data-loggers are being used here to monitor near real-time surface movement, rainfall, and groundwater pressures in the landslide (Massey & Palmer 2007).

**Cause of the Landslide**

The Abbotsford Landslide occurred in an area with a history of slope instability (Benson 1940; Gordon 1960). By present-day standards that association would call for greater vigilance in development of the area, particularly expansion of the sand pit adjacent to the relict Sun Club Slide at the base of a slope. That landslide had been identified when the DCC pipeline route was evaluated (Benson 1945), and was again noted during investigations for the motorway (Gordon 1960). It was unfortunate and perhaps ironic that most of the sand removed from the toe of the slope, thereby reducing its stability, was used to stabilise a nearby landslide reactivated by motorway construction.

The Commission of Inquiry supported the DSIR’s conclusion that the presence of very weak clay layers containing montmorillonite within a 7–10º dip slope was the basic precondition for the landslide, with increased rainfall and two human actions – the sand quarry and the leaking DCC water main – identified as the main contributing factors. Although groundwater levels and the volume of water leakage could not be precisely determined before the slide, the overall effect of the sand excavation was that the water table in the area had to rise about 0.3 m less in order to reach the critical stability condition at the time the slope failed on 8 August. The sand excavation reduced stability of the overall slope by about 1%, and except for small local movements in one area at the toe, the slope remained essentially stable for ten years after the excavation was completed.

This suggests that the sand excavation did not trigger the onset of slope movements in September 1978 which culminated in the final movement on 8 August 1979. Raised groundwater levels are inferred in the slide area due to increased long term rainfall, rupture of pipes within the slide area, and leakage from the water main at the top of the slope. Leakage
from the water main alone was possibly equivalent to about a 40–50% increase in rainfall over three years prior to the landslide. Therefore, in the author’s opinion, a long term rise in groundwater levels in the landslide area controlled the timing of the failure, and in this sense is believed to have triggered the final movement of the Abbotsford Landslide on 8 August.

**CONCLUSIONS**

The Abbotsford Landslide is the most damaging single landslide to have occurred in an urban area in New Zealand. After 28 years, it still provides a valuable and timely reminder of the potential dangers of excavations at the toes of slopes, leaking water pipes in urban areas, the need for investigations to assess landslide hazard and risk, and appropriate responses by the authorities to manage and reduce the risk of property damage and loss of life.

Geotechnical investigations enabled the extent, and nature of the risk presented by the Abbotsford Landslide to be determined as it developed over the two months prior to the final movement on 8 August 1979. However, because ground movement data were overlooked in the two days prior to the failure, the area was not completely evacuated when the collapse occurred. Undeniably, the people who remained on the slide mass were fortunate to survive.

In retrospect, it must be concluded that the assessment and management of the Abbotsford Landslide event was much less than perfect. This case history underlines the value of plotting and analysing landslide movement data as soon as it is collected. It also shows that human safety should be the prime consideration in landslide risk assessment, and it is generally better to make decisions based on the “worst case” rather than the most likely outcome.

**REFERENCES**


Investigation Report.


ACKNOWLEDGEMENTS

The author wishes to acknowledge the contributions made in 1979 and 1980 by his DSIR colleagues to the reports on which the paper is based, especially Graham Bishop, Ian McKellar, Pat Suggate, Graham Salt, Roy Northey, Peter Barker, and Nick Perrin. His present GNS Science colleagues Grant Dellow and Dick Beetham are also thanked for reviews of this paper.
REPEATED COLLAPSE OF CUT SLOPES DESPITE REMEDIAL WORKS

S. G. Lee
Department of Civil Engineering
The University of Seoul, Seoul, Korea

S. R. Hencher
Halcrow China Ltd.
Department of Earth Sciences, University of Leeds, UK

Abstract: The collapse of slopes and the subsequent costs of remedial works are often the result of insufficient geological investigation and inadequate interpretation of ground conditions prior to design. This is compounded by inadequate safety inspections and systematic problems associated with poorly defined responsibilities for the stability of cut slopes. This paper reviews such problems in detail with reference to large slope failures in two areas in Korea.

INTRODUCTION
Expenditure on ground investigation is typically low in Korea, of the order of 0.05-0.2% of the total construction costs; ground investigation costs for cut slope work are generally even lower. Low cost, low quality investigation means that geological structure is rarely recognized as an important factor for the stability of cut slopes at the design stage. Related statistics are that almost 30% of cut-slopes collapse during their formation and, following an in-depth review of ground investigation and design-related items for 102 designs for cut slopes, only 10% were found to have been carried out using appropriate methods (Lee & Hencher 2007; Lee et al. 2007). It is estimated that there are about 1 million cut slopes in Korea which is of the order of 20 times the number in Hong Kong (Lee et al. 2008).

Standards for geological investigation and design are not generally well-specified in Korea and this contributes to the difficulties in identifying the responsible party for any particular collapse despite apparent poor design. Furthermore as investigations following failures are also often inadequate, causes of collapse are not properly identified and failures are therefore attributed incorrectly to some unforeseen natural cause. In addition, where the responsible party remains unidentified, it is common that remedial works have to be paid for by the Client/Owner regardless of the cause of collapse and the cost of the remedial works. Given that failures are generally attributed to natural causes, losses and injuries from cut slope collapse are often not indemnified despite legal action being taken in many cases.

One of the ironies in the attribution of slope failures to natural causes (generally heavy rainfall) is that cut slope design standards of Korea generally require that groundwater is assumed at the ground surface which should account for the most severe condition - it should not be possible to put the blame on excess water pressures as the result of abnormally severe rainfall. Conversely therefore, failures might generally be attributable perhaps to inadequate geological investigation and incorrect ground model, wrong geotechnical parameters or errors in slope stability analysis. These difficulties in identifying contributing factors in cut slope design and subsequent failure has become a matter of public concern in Korea. KBS TV,
Korea’s public service broadcaster, has presented two documentaries entitled “Money is earned only after collapses” (May 2001) and “Collapses attributable to the sky?” (September 2002). Two examples of large cut-slope collapses, which illustrate the problems concerning cut slopes in Korea, are discussed below (Figure 1).

CASE 1: ROAD CUT NEAR TUNNEL PORTAL AREA OF THE WHANGRYEONG MOUNTAIN IN BUSAN

Overview
Busan is the second biggest city in Korea with a population of 4 million in an area of 760km$^2$. Forty three percent of the area is mountainous and is classified as a “cut-slope disaster risk area” (Lee 1999). In September 1999, tension cracks of 20-30m deep and 15m wide were found 80m behind a large scale cut slope (50m high and 130m wide) located at the entrance of Busan Tunnel where 80,000 vehicles are passing through per day. The related failure of the cut slope involved 140,000m$^3$ of material that moved 15m; several vehicles were buried and 130m of bridge piers were destroyed. The local geology comprises inter-bedded layers of strong sandstone and weak tuffaceous shale with bedding dipping at 15-20° (Figure 2).
Process of Development

Original Design (July 1991)
Boreholes were not put down specifically at the cut slope for the original design (Yoo 1991); a borehole investigation (terminated 1m into moderately weathered (MW) rock) was conducted for bridge foundations 30m away from the cut slope. Upper soils were only 1.2-1.7m thick. Underlying the soil, MW rocks were collected as rock fragments; TCR \( \leq \) 10\%, RQD = 0 (Figure 3). The 40m high slope was then designed with a 1:0.5 (63°) standard gradient, based on the Korean construction standards (KEC 2001; MOCT 2004) for the design of cut slopes as follows: moderately weathered to fresh rock 1:0.5 (63°), highly weathered rock 1:1 (45°), and soil 1:1.2 (40°) to 1:1.5 (34°) depending on rock material strength.

1st Safety Inspection (January 1993)
Field mapping was carried out after a small scale 1st collapse before completion of cutting.
work and the geological conditions are described in some detail in Kim (1993). Vertical joints were commonly developed within bedding units; thickness of sandstone layers was typically 300-400mm with thin (10-20mm) highly weathered (HW) intercalated shale layers. Stereographic projection analysis indicated possible toppling failure; the upper part of slope was therefore redesigned with a gentler gradient of 1:0.6 (59°) (Figure 4).

Figure 4: Result of 1st cut slope safety inspection (reduced gradient of the upper part to 1:0.6).

2nd Safety Inspection (December 1995)
Further collapses of the cut-slope occurred infrequently after completion in May 1995 and, according to the field geological survey conducted for the safety inspection in December 1995 (Chae 1995), some small-scale wedge mechanisms were identified but the risk of bedding plane translational failure was not analyzed. The possibility of circular failure was also considered high (Fs = 0.99), so for stabilization, it was suggested to reduce the gradient of the lower part of the cut-slope gentle to 1:0.6 (59°) (Fs = 1.14). In addition, the surface of cut slope was to be covered with wire-mesh and vegetated shotcrete (Figure 5).

Figure 5: Result of cut slope stability review in 2nd safety inspection. (a) MW slope by 1st safety inspection (1:0.5), (b) MW slope by 2nd safety inspection (1:0.6) and (c) status of reinforced cut-slope after 2nd safety inspection (before 3rd collapse)

3rd Safety Inspection (November 1999)
Heavy rain occurred in September 1999. The accumulated rainfall for 2 days before failure was 100mm; maximum rainfall per hour on the day of the failure was 39mm (KMA 1999). Such rainfall intensity is considered dangerous according to landslide triggering data of Korea (KFS 1993). A 3rd large scale collapse developed along a bedding plane inclined at 15-20° and filled with clay. This failure resulted in the burial of 10 vehicles and destruction of a bridge (Figures 6 & 7). As this important road was to be re-opened urgently, a temporary
protective wall was installed using H-piles and landslide debris removed (toe-cutting); collapses occurred continuously in October 2000 and January 2001 during the works.

At the 3rd safety inspection by a professional institute (Han 1999), boreholes were put down at 8 locations to 22-35m depth and direct shear tests carried out on clay samples which gave reported strengths of $\phi = 21.4^\circ$, $c = 11kPa$ at natural moisture content and $\phi = 16.8^\circ$, $c = 11kPa$ where saturated.

The failure mechanism was identified as bedding plane slip involving clay infill. A more gentle gradient (1:2.0 (27°)) was proposed together with reinforcement H-piles to provide $F_s = 1.4$ which is higher than $F_s = 1.2-1.3$ normally adopted because of the importance of the slope (Figures 8 to 10).

![Figure 6: Scene soon after major landslides. (a) Side view and (b) front view](image)

![Figure 7: Geological characteristics of failure planes. (a) Failure plane from side, (b) bedding plane from the bottom, (c) tension cracks at the top, (d) bedding planes exposed in tension cracks.](image)

![Figure 8: Stereographic projection for fracture data - mostly bedding and orthogonal joints (Choi & Paik 2002).](image)
Figure 9: Photograph of core of HW shale with clay filling developed at 26 m deep in MW-SW sandstone (Han 1999)

Figure 10: (a) Result of 2nd safety inspection and (b) scene of completed remedial works

Discussion

(1) The fundamental problem was that a proper ground investigation was not performed for this large, important cut slope and that boreholes were taken to an insufficient depth at 30 m. Data from the investigations for nearby structures were totally inadequate for design of this slope.

(2) Once the nature of the geology had been broadly established (it was not difficult), the application of kinematic analysis was poor, totally overlooking the possibility of translational failure on low angle but weak bedding surfaces and poor understanding of the limitations of the stereographic projection method (Hencher 1987; Hencher & Knipe 2007).

(3) Rock cut slope collapses are commonly the result of infiltration of groundwater into tension cracks developed to the rear of cut slopes; analysis of aerial photographs shows that tension cracks were already developed at the back of cut slope by the time of 2nd safety inspection in 1995 (Figures 11 & 12). The failure type of the cut slope was misjudged as a circular failure of soil rather than translational bedding plane slip in the 2nd safety inspection despite the Fs for such a mechanism to be low as 0.84 when fully saturated. A contributing factor to the history of collapse was that remedial works were carried out in a hurry with no identification of the basic cause.
Figure 11: Process of collapse from original design to final catastrophic failure. 
(a) Original ground before cutting work, (b) after cutting work (1995), 
(c) initial development of tension cracks at the back of rock masses 
(1995-1999) and (d) creation of large landslide (1999).

Figure 12: Aerial photographs of the landslide area before the major landslides in September 1999; (a) 15/11/93 (b) 19/11/96 (c) 23/11/97 (d) 17/06/98.

CASE 2: CUT SLOPE AT THE DAEMO PRIMARY SCHOOL IN SEOUL

Overview
Seoul, Korea’s capital has a population of 10 million people in an area of 605 km², 20% of which is mountainous. A primary school for 1,700 students was constructed on a cut platform in the gneissic area in Seoul. The 1st safety analysis and landslide preventive works were performed following a collapse soon after construction of the slopes; a 2nd safety assessment was carried out because of continuing concerns regarding the stability of these cut-slopes adjacent to a school (Figure 13).
Process of Development

Original Design (May 1994)
For the original design in 1994, no boreholes were carried out despite the large scale of cutting (20–30m high slopes, 250m wide). Instead, borehole data 300m away from the site was assumed to be relevant; the area was considered to comprise completely weathered (CW), HW, and MW rock. The lower part of the cut-slope was designed to have a 1:0.5 (63°) standard gradient with vegetated surfaces (Cho 1994) (Figure 14).

1st Safety Inspection (November 1998)
Part of the cut slope collapsed during heavy rainfall in July 1998. The failure apparently occurred along the boundary between soil and MW rock (Figure 15). One borehole was proposed at the location of the collapse to terminate up to 2m into slightly weathered rocks. Rock bolts were proposed in some sections considered prone to planar failure. Otherwise, the lower part of the cut slope was considered safe with a 1:0.5 (63°) gradient. The upper part of the cut slope was proposed to be cut at a gradient of 1:1 (45°) (Park 1998) (Figure 16).
Figure 15: Cut slope collapses in August 1998. Front view after landslide (Lee 2001)

Figure 16: Location of boreholes and cross section following 1st safety inspection. (a) Location of borehole investigation and (b) cross section.

2nd Safety Inspection (November 2000)
Works were completed following the recommendations from the 1st safety review of the collapsed area but residents near to the site made civil appeals to Seoul Metropolitan City Council and a 2nd review was instigated for the entire cut slope (Lee 2000).

Despite much of the cut slope being covered with vegetation, exposed areas of slope were mapped geologically (Figure 17). In addition, 13 boreholes were put down to the height of the cut slopes and logged using TV cameras (Figure 18). In addition, 2 lines of field seismic refraction survey were performed at the ground surface of the upper part of the cut slope. A 3D stratum model (Figure 19) was created through integrated analysis of the surveyed data including the videos from the boreholes. The investigation allowed the joint fracture network to be characterised and several faults were located. It was also found that the boundary between soil and rock in the whole cut-slope area was inclined towards the school. Shear strength tests were performed on the boundary between soil and rock together with a variety of other laboratory tests at natural and saturated moisture conditions (Lee 2001).

Using numerical analysis by DEM (distinct element method), failure mechanisms involving discontinuities were found to be relatively stable. Displacement analysis by FDM (finite difference method) showed that displacement was greatest at the boundary plane between CW and MW rock (Figure 20). Considering both translational and circular failure mechanisms along the boundary of CW and MW rock by limit equilibrium, it was
demonstrated that there was a concern for stability under high groundwater conditions (-3m from ground surface which equates to a 100-year frequency 3-day consecutive rainfall analysis). It was therefore proposed to reduce the gradient (1:1) and apply anchoring/nailing to satisfy both circular failure and planar failure conditions (Figures 21 to 23).

Figure 17: Panoramic photograph of Daemo primary school slope (Lee 2000)

Figure 18: Results of borehole investigation. (a) Fault zone observed by borehole camera (BH-9) and (b) vertical section view interpreted from two boreholes (Lee & Geum 2002).

Figure 19: 3D geological models based on borehole camera interpretation (Lee & Geum 2002)
Figure 20: Result of numerical analysis of cut slope stability by (a) DEM and (b) FDM (Lee 2000).

Figure 21: Original slope (at the highest water level). (a) Circular failure in soil and (b) plane failure along the boundary of soil and rock (Lee 2000).

Figure 22: Cut slope (1:1, at the highest water level). (a) Circular failure in soil and (b) plane failure along the boundary of soil and rock (Lee 2000).
Lessons
It is inappropriate to carry out design solely on the basis of borehole data from a distance of 300 m from site as was done for the original design. It was inappropriate, as was originally done to only consider circular failure within the upper soil as a mechanism for failure. The potential for more extensive circular failure should have been considered continuing into the lower stratum of MW rocks with RQD = 0%. Other potential modes of failure should also have been considered; in this case the original failure occurred on the essentially planar interface between soil and rock and indeed additional preventive works needed to be designed for that failure mode as demonstrated by the additional investigations and analysis (Lee 2002; Lee & Hencher 2007).

CONCLUSIONS
In general, the mountainous areas in Korea have relatively favourable stability conditions because soils are thin and the rocks are not deeply weathered. However, inattention to geology during investigation and the lack of proper design standards means that, with the cutting of more and more large scale cut slopes as mountain areas become industrialized, the failure of large cut slopes is an increasing trend. As the standards for geological survey and design are not properly specified and inspections performed after failures are often insufficient, the causes of collapses are not properly identified. Most are attributed to “natural disaster”. As a consequence, the costs of remedial works are paid for by the Korean government and those affected by failures are not indemnified in the majority of cases. It is of prime importance that the system is improved with clear identification of responsibilities for design, construction, and guidance for inspections. Guidance is also required for appropriate geological survey with technical training and the development of consistent technological systems for geological survey, design, construction work, and maintenance. Attention is also required for management with interaction between those responsible for maintaining the stability of cut-slopes. There is a need for establishing a database of cut slope information and an integrated management system.

REFERENCES


Ministry of Construction & Transportation (MOCT) (2004). A guideline for the design of national roads.


**ACKNOWLEDGEMENTS**

The Authors would like to thank Korean geotechnical engineers who reviewed this paper. This research was supported by a grant (NEMA-06-NH-05) from the Natural Hazard Mitigation Research Group, National Emergency Management Agency, Korea.
THE STOREGGA SLIDE – CASE STUDY OF AN OFFSHORE MEGASLIDE IN THE NORWEGIAN SEA

Farrokh Nadim and Tore J. Kvalstad
International Centre for Geohazards
Norwegian Geotechnical Institute

Abstract: The Storegga slide occurred 8200 years ago and was the last megaslide in the Norwegian Sea where similar slides have occurred with intervals of approximately 100,000 years since the onset of continental shelf glaciations about 500,000 years ago. A geological model for the Plio-Pleistocene of the area explains the large scale sliding as a response to climate variations. The regional seismic stratigraphy indicates that sliding occurs at the end of a glaciation period or soon after the deglaciation. The Storegga slide and earlier slides that have occurred in area are in general translational with the failure planes related to strain softening behaviour of marine clay layers. The destabilisation factor leading to the slide was related to rapid loading from glacial deposits with generation of excess pore pressure and reduction of the effective shear strength in the underlying clays. Basin modelling has shown that excess pore pressure generated in the North Sea Fan area was transferred to the Storegga area, and this process was detrimental to the slope stability situation in the old escarpments in distal parts of the Storegga slide. The slide could have been triggered by this process in combination with a strong earthquake in an area 150 km downslope from the Ormen Lange gas field, and developed as a retrogressive slide.

INTRODUCTION
One of the largest submarine slides in the world, the Storegga slide, occurred about 8200 years ago in the Norwegian Sea (Bugge 1983; Bryn & Andersen 2007). Figure 1a shows the location and the approximate extent of the Storegga slide. The estimated soil masses removed by the slide were between 2500 and 3500 km$^3$. Evidence of a tsunami generated by this slide event has been found along the coastlines of Norway, Scotland and the Faeroe Islands. Figure 1b shows an exaggerated bathymetric picture of the central, upper part of the slide showing its upper and lower headwalls. The upper headwall scar has a total length of about 300 km. The headwall slide scars are steep (locally about 30°), with heights in the order of 100 to 250m. The main part of the failure surfaces of the Storegga slide followed the stratigraphy at inclinations of less than 1.5°, with large areas having an inclination of 0.3 to 0.5°. The understanding of the in situ conditions and mechanisms that could generate such an enormous slide at such low average inclinations is of vital importance for the assessment of the risk for future large-scale sliding in the area.

In 1997, the Ormen Lange gas field was discovered within the slide scar of the Storegga slide (Figure 1b), about 130km west-northwest of the city of Kristiansund in Norway. With estimated recoverable gas reserves of 400 billion Sm$^3$, Ormen Lange is the second largest gas field in the Norwegian Sea. Norsk Hydro is the operator for development and construction of the Ormen Lange field. Considering the enormity of the Storegga slide and the potentially catastrophic consequences of a similar event today, it was essential to clarify and quantify the risks associated with submarine slides in the area to obtain approval for field development from the authorities. A number of studies were therefore initiated by Norsk Hydro to assess the present sea floor stability situation and the risk for future debris flows into the field.
development area or pipeline corridors. These studies required detailed evaluation of all available geotechnical, geological and geophysical data. The main questions that needed to be answered were:

- How could a slope with inclination of 0.5° - 2° fail and how could the failure develop to a gigantic submarine slide like the Storegga slide?
- What was the triggering mechanism and where was the slide initiated?
- Is the probability of occurrence of a new large slide too high for the development of the Ormen Lange field?

Obviously, a major part of the work was developing explanation models for the Storegga slide. The paper summarises what has been learned about the Storegga slide through the numerous studies that were carried out during the past decade to answer the above questions.

Figure 1: (a) Map showing the location and extent of the Storegga slide. (b) Central and deepest cut part of the Storegga slide scar (Ormen Lange gas field in blue)

REGIONAL GEOLOGICAL SETTING

Figure 2 compares the bathymetry model of the mid-Norwegian continental margin with deeper structures and illustrates how deeper structures control the present-day seabed physiography. The seabed physiography is the result of the interaction between these structural influences and the patterns of erosion and deposition from mid- to late Cenozoic times. The major events that have exerted sedimentary control over that period can be summarised as follows (Bryn & Andersen 2007):

- Eocene to earliest Miocene times: Sea floor spreading initiated between Norway and Greenland (Bukovics & Ziegler 1985). Once spreading began, the area became quiescent and subsided rapidly, allowing accumulation of fine-grained oozes and shales of the Brygge Formation (Figure 4) in relatively deep water.
- Mid-Miocene to early Pliocene times: Formation of basalt plateaus and basin structures such as Helland-Hansen Arch, the Ormen Lange Dome and the Vema Dome (Eidvin & Rundberg 2005). Fine-grained sediments trapped on and around these structures, increasingly under the influence of currents, as ocean circulation patterns became established in response to continued opening of the of the Norwegian-Greenland Sea (Thiede & Myhre 1996). The basin structuring controlled the subsequent slide history.
Late Pliocene to mid-Pleistocene times: Pronounced shelf edge progradation due to increased erosion caused by uplift of the Norwegian mainland and formation of ice cap in central areas (Riis 1996). Sediments were transported and deposited as a huge prograding wedge on the shelf, however, little material reached far into the basin. The first sliding started to occur, possibly due to glacial activity.

Mid-Pleistocene to recent times: Climate change became more pronounced and glaciations became more frequent during this period (Henrich & Baumann 1994), resulting in alternating units of glacial debris and fine-grained clays deposited by contour-following currents (i.e. contouritic deposits). The cyclic nature of these strata is believed to play a key role in creating slope instability (Bryn et al. 2005a).

Slope Stability and Palaeo-slides
Palaeo-slides are ancient slides that have been detected either at the seabed or within the sub-surface. The most readily identifiable palaeo-slides are those that leave visible scars on the seabed, for example the Storegga slide. Using seismics, it is possible to detect older palaeo-slides, such as the Tampen Slide on the North Sea Fan (Figure 4), that have been infilled and covered by sediments. Identifying the historical pattern of slope failure is essential to an understanding of the evolution of the continental margin and of the timing and causes of slides. The Storegga slide was indeed a mega-event, but it is important to bear in mind that it represented only the latest mass movement in a long history of slope failures on the mid-Norwegian margin. It must be stressed that the buried palaeo-slides identified in the regional study are necessarily the larger events, commonly covering areas of several thousand square kilometres (the scar of the Storegga slide covers an area larger than Belgium) and removing several cubic kilometres of sediments. Smaller slides, which could nonetheless be devastating to any installations on the seabed, may not have been mapped due to a combination of limited data coverage and seismic resolution.
The combination of continental uplift and ice-cap development in the late Pliocene led to increasing erosion rates. Large quantities of sediment were transported from the mainland and deposited on the outer shelf and upper slope, and during the Late Pliocene/Early Pleistocene, the shelf edge moved to its present-day location (Rise et al. 2004). From this time on, the glaciations extended repeatedly to the shelf edge and during the short periods of glacial maxima (ca. 10,000 years), large volumes of sediments were deposited on the continental slope in front of the ice-streams. The Storegga slide scar is located in the depression between two main glacial depocentres.
The Storegga slide and older palaeo-slides display several similar features, indicating similar slide-mechanisms, slide development and relationship to the regional geology (Bryn et al. 2005b). The typical common characteristics are:

- Strata-parallel slip surfaces are found in seismically stratified marine clay units.
- The slip surface jumps among different stratigraphic levels, stepping up the slide scar, in particular when approaching the headwall of slide.
- Rotated slide blocks are found near the headwalls with modest lateral displacement and little internal remoulding.
- The headwalls seem to have been stable during the time needed to fill in the slide scar.

**IN SITU CONDITIONS**

The present in situ conditions have been established on the basis of the information gathered through geological, geophysical and geotechnical investigations in the area. The geotechnical information were gathered from the Seabed Project (Britsurvey 1997, 1999; NGI 1998) in the initial phase and from the site-specific soil investigations performed in connection with the Ormen Lange field development in 1999, 2000, 2001, and 2002. Piezometers were installed in 4 deep boreholes in 2001 and pore water pressure readings have been gathered (Tjelta et al. 2007).

![Figure 5: Location of borings (blue points) and exploration wells (left), and stratigraphy and projection of borings into idealised profiles of the deep and the shallow part of the Storegga slide (right). Piezometers were installed at Sites 19_2, 20, 22 and 99](image)

Figure 5 (left) shows locations of borings and installed piezometers in the area surrounding the Ormen Lange field. Further borings were carried out in more distal positions in the slide area and to the north of the Storegga slide. An idealised projection of all borings and a principal summary of the main stratigraphy are also presented in Figure 5 (right).

**Bathymetry**

The water depth is in the order of 250 to 300 m at the shelf edge, increasing rapidly down the upper headwall of the Storegga slide to about 500 to 600 m. The area between the upper and the lower headwalls is covered with debris from the Storegga slide event and the very rugged surface of slide blocks and remoulded debris is gradually being infilled with Holocene marine clays (Figure 6). This area is inclined slightly (about 1°) towards WNW over a distance of about 20 km down to the edge of the lower headwall at about 900 m water depth. The lower headwall falls steeply (10 to 30°) down to about 1100 m water depth.
Figure 7 shows a profile through the slide area from the shelf edge to about 2500 m water depth. The average slope from the top of the upper headwall to the abyssal plain at 2800 m depth is less than 0.6°. Over large parts of the area the inclination of the major slip surfaces is less than 0.4°.

Figure 6: Example of bathymetric view of upper headwall (left) and seabed topography in area between the upper and lower headwalls showing large slide blocks with infill of Holocene clay (right)

Figure 7: Seabed slope through slide area from shelf edge to about 2800 m water depth

Depositional Processes
The geological conditions in the area are of vital importance for interpretation of the soil conditions. The soils layers of interest were deposited mainly during the last 900,000 years and were thus controlled by the cyclic (about 100,000 year period) glacigenic depositional processes in the area. Climate variation controlled the eustatic sea level changes, ocean circulation pattern, and sediment supply to the shelf and the shelf edge. The depositional sequences were formed in response to waxing and waning glaciers across the continental shelf.

During peak glaciations, the continental shelf was influenced directly by grounded glaciers. The main material transport to the shelf edge was provided by the fast flowing ice streams transversing the shelf. The marine clay deposits on the shelf were mostly thin and easily eroded. Thus, the shelf consists mainly of glacial tills, locally with thin embedded layers of marine clay.
Large amounts of glacial debris were deposited at the shelf edge or pushed over the edge, giving rise to frequent debris flows down the continental slope during peak glaciations. During the much longer periods between peak glaciations, normal marine and/or distal glacial marine deposition prevailed, producing stratified fine-grained sediments with higher water contents (Berg et al. 2005). This gave rise to a sequence of interbedded layers of marine clays (with high clay content and open grain structure) and glacial debris material, mainly remoulded and reworked materials with lower clay content. The marine clays are softer and more sensitive (i.e. lose more of their strength when deformed) than the glacial material, and are sometimes referred to as hemipelagic or contouritic sediments in the literature. Figure 8 shows a long seismic section crossing the upper and lower headwall in the Storegga slide with interpreted seismic stratigraphy. The rugged surface of slide deposits is clearly seen.

**Stratigraphy and Main Soil Types**
The intact soils can be roughly divided in two types: marine clays with clay content in the range of 45 to 65% and plasticity index greater than 25%, and glacial deposits (debris flows and tills) with clay content of about 30 to 40% and plasticity index of 12 to 25%. The dominating clay mineral is illite, with smaller amounts of chlorite, kaolinite and smectite. The glacial deposits contain also gravel and stones, which hampered the soil investigations considerably. Figure 9 gives an idealised summary of soil properties through the different strata. Marine layers, O3, R2 and S2, are the main slide base layers, with higher clay content, water content, plasticity index and liquidity index, and correspondingly with lower strength parameters than the glacial deposits.

![Figure 8: Seismic profile crossing the deep slide scar from upper through lower headwall with interpreted stratigraphy (two-way time versus shot point)](image)

The large area involved and the limited number of borings required a thorough review of all available data. Due to the unloading in the slide scar area and glacial loading of some layers in the shelf edge area, there was a considerable spatial variation in the overconsolidation ratio. Furthermore, the uncertainty in pore pressure conditions had to be taken into account and also the uncertainty in thickness of overburden lost.

The loss in overburden in the slide scar has been estimated mainly by interpretation of cone penetration tests (CPTs) and oedometer tests. Along the upper headwall, the bathymetry and seismic profiles give a clear picture of the loss in overburden.
In Situ Pore Pressure Conditions

The pore pressure measurements (Tjelta et al. 2007) have been of great value for assessment of the effective stress conditions in the headwall area as well as in the slide scar. The high sedimentation rates in the area indicated that excess pore pressure could exist. An analysis based on estimated geological ages of the different strata, the observed thickness and oedometer test data was performed and indicated excess pore pressure conditions prior to the Storegga slide and at present in the upper headwall area where no unloading has taken place. The uncertainty connected to this type of analysis will always be considerable due to variations in drainage conditions. Thus the installation of piezometers and the following readings of pore pressure provided valuable proof.

Figure 10 shows the measured pore pressure values for Sites 19_2, 20 and 22 against depth below sea level. The hydrostatic and the total overburden stress distributions are shown and so is the calculated pore pressure distribution for Site 20. It can be seen that there is a moderate excess pore pressure at these three sites. Compared with hydrostatic pressure measured from seabed level, the excess pressure is in the range 8 to 17%. In the deeper part of the slide scar where the loss in overburden is estimated to be about 250 m, hydrostatic pore pressure was measured.

POSSIBLE STOREGGA SLIDE SCENARIOS

Slide Scar Morphology

The Storegga slide has an upper headwall length of 320 km, and the slide narrows to a 60 km wide gateway in opening between the Møre and Vøring volcanic highs (Figure 2). In addition to the upper headwall, the Storegga slide scar contains several headwalls and scarps (Figure 8).

The lowermost headwall was probably formed in the Brygge formations by evacuation/mobilisation of oozes during the late Pliocene/early Pleistocene (Rise et al. 2004).
The sediment thickness increases upslope and causes higher compaction and sediment strength. The scarps mark glide plane jumps to higher stratigraphic levels during the retrogressive slide development. The northern and southern parts of the slide failed in the marine clay layer of sub-unit O3 with a depth of less than 100 m below the pre-slide seabed (Figure 8). Lateral spreads are the most common morphological feature observed in these areas. These features appear as parallel ridges and troughs and are formed by extensional displacement of the sediments overlying glide planes or a slip zone in the hemipelagic/contouritic sediments. In the central part of the slide including the Ormen Lange area, the detachment has followed deeper sub-units. Failure has taken place in the S2 sub-unit below the lower headwall and the R2 sub-unit in the Ormen Lange area (Figure 8). Blocky debris flows dominate the morphology in the central slide scar with lateral spreads towards the upper headwall. The R-headwall may have reached 500 m high in the Ormen Lange area during the slide, based on a reconstructed pre-slide topography. Compression ridges of varying sizes are present over large parts of the central slide area, but the most prominent compression zone is in the north-western part of the North Sea Fan (Figure 2). The main detachment was in the S2 sub-unit with a region of shallow compression in the O3 sub-unit at the perimeter towards the North Sea Fan. The compression zone is most likely the result of collapse of the glacial fan built out in the Ormen Lange area.

The history of previous sliding and evacuation events (Rise et al. 2004) left old slide scars with headwalls that provided locally steep slopes in the lower to mid slope region, which probably only had a thin drape of sediments prior to the Storegga slide. Further upslope, the old slide scars and headwalls were deeply buried by subsequent sedimentation. The slide scar and the lateral spreads have similarities with onshore slides in quick clay or liquefied sandy/silty sediments. The observed pattern in the Storegga slide scar is related to the strain softening behaviour of the marine clays and numerical modelling has demonstrated that a
retrogressive slide process was likely despite the low slope gradients (~1º) in the Storegga area. (Kvalstad et al. 2002).

**Soil Shear Strength Evaluation for Stability Analyses**

The stress history in the slide area is complex and had to be considered in the evaluation of strength parameters for the stability analyses for the present situation as well as for the explanation of the Storegga slide. The following points required special attention:

- The effect of excess pore pressure prior to the slide event and in the upper headwall.
- Overconsolidation from unloading in the slide scar and also from glaciers at the shelf edge.
- Strain dependency of strength parameters (drained and undrained strain softening).

These had the following impact on the strength models:

- The undrained strength model should take excess pore pressure during consolidation phase prior to slide into consideration, then the effect of loss of overburden and the present excess pore pressure regime.
- Undrained strain softening from peak strength to residual and remoulded strength should be described for explanation of undrained progressive failure, mobility required for retrogressive failure and modelling of slide run-out after failure.
- The drained strength model should cover stress dependency and overconsolidation effects on the strength parameters (non-linear failure envelope).
- Drained strain softening from peak towards residual friction angles should be evaluated with consideration of the effects of clay content and clay mineralogy, which are known to have a major impact on the residual strength.

The undrained strength from triaxial compression and extension tests and direct simple shear tests were thoroughly evaluated using the SHANSEP (Stress History and Normalized Soil Engineering Properties) approach (Ladd & Foott 1974). SHANSEP is a generalised model of undrained shear strength that accounts for the effect of overconsolidation. The normalised undrained strength (with respect to consolidation stress) is expressed by the following relationship:

\[
\frac{s_u}{\sigma'_v}_{OC} = \left( \frac{s_u}{\sigma'_v}_{NC} \right) \cdot OCR^m
\]  

[1]

where \(s_u\) = undrained shear strength, \(\sigma'_v\) = effective vertical consolidation stress, OCR = overconsolidation ratio, \(m\) = exponent. The subscript OC means overconsolidated, while the subscript NC means normally consolidated. Eq. [1] can be rewritten as:

\[
\log \left( \frac{s_u}{\sigma'_v}_{OC} \right) = \log \left( \frac{s_u}{\sigma'_v}_{NC} \right) + m \log(OCR)
\]  

[2]

Eq. [2] can be plotted as a linear function in a \(\log(s_u/\sigma'_v)_{OC}\) vs. \(\log(OCR)\) diagram, crossing the vertical axis at \(\log(s_u/\sigma'_v)_{NC}\) with inclination, \(m\).

Figure 11 shows a plot of triaxial (CAUC) compression test data sorted according to layer...
type and allowed determination of the exponent, m, to be close to 0.7 with \( (s_u/\sigma'_v)_{NC} \) – values of about 0.31 for the glacial clays and 0.27 for the marine clays.

Similar treatment of DSS test data gave nearly the same exponent and lower \( (s_u/\sigma'_v)_{NC} \) – values. The strength anisotropy was evaluated by comparison of samples subjected to similar consolidation stresses in compression direct simple shear and triaxial extension giving \( s_u^{DSS}/s_u^{C} = 0.8 \) and \( s_u^{E}/s_u^{C} = 0.7 \).

The triaxial and DSS shear tests were generally run to high shear strains. Strain softening was most pronounced for the marine clays which exhibited a reduction of about 30 to 50% of the peak strength when sheared to more than 20% axial strain while the glacial deposits in general showed little or no strain softening.

**Remoulded Strength and Sensitivity**

A large number of fall cone tests were performed on remoulded material. Sensitivity is normally expressed as the ratio between the undrained shear strengths measured on undisturbed and remoulded material (at the same water content) with the fall cone test. Remoulded strength is not applied directly in the stability analyses. However, for analysis of mobility of debris in a retrogressive slide, the remoulded strength is considered to give a lower bound estimate of strength during the first part of the debris run-out process. With increasing run-out distance, a gradual mixing-in of water may take place leading to even further reduction in strength of the debris flow material.

**Drained Strength**

Peak strength was determined from triaxial and direct simple shear tests (Figure 12). A clear effect of stress level and overconsolidation can be seen. For the steepest slopes in the unloaded (and thus overconsolidated) headwalls, critical slip surfaces for drained, long-term failure are located close to the surface with low normal stresses.

Residual friction angles have been determined from ring shear tests mainly on remoulded reconsolidated material and seem to correspond well with the lower bound values from triaxial and DSS test at large shear strains. Residual friction angles are heavily dependent on the clay mineral composition and system chemistry, mainly salt content in pore fluid and to a lesser extent on the stress level. Figure 11 shows the compilation of the lowest friction angles (large strain values) from the triaxial CAUC tests and the results of ring shear tests.

**Slope Destabilisation and Slide Triggers**

The following destabilising factors were evaluated as potential triggers for the Storegga slide: effects of high sedimentation rates during peak glaciations, gas hydrate melting, gas charging of shallow sediments, diapirism and earthquakes. A common feature of all these processes is that they increase the pore pressure in the sediments and decrease the effective soil strength.
Sediment Loading

During the last peak glaciation (the Late Weichselian), the sediment input to the Storegga area was modest, reaching approximately 100 m in the upper part of the slope. In comparison, approximately 500 m of sediments accumulated in the depocenter of the North Sea Fan (Figure 2). During Saalian glaciation, the main deposits seem to be the glacial sediments in the Ormen Lange area (Figure 2). A similar thickness was probably deposited in the North Sea Fan, but later partly eroded by the Tampen Slide (Solheim et al. 2005). The distal parts of the Storegga slide are assumed to have experienced low sedimentation rates during the last 300,000 years. Rapid loading of sediments with low permeability may cause development of excess pore pressure when the length of drainage path increases faster than the time required for consolidation. Thus pore over-pressure was created in the contouritic marine clays and ooze, with then highest over-pressure in and close to the glacial depocenters. Numerical modelling indicate potential excess pore pressures in the order of 20 to 30% due to loading in the upper slope and shelf edge at the time of the Storegga slide (Kvalstad et al. 2005). The measured recent excess pore pressure 2 km behind the upper headwall was 15 –17% of the hydrostatic pore pressure (Tjelta et al. 2007).
Numerical modelling of possible load effects from the glacial deposits in the North Sea Fan was carried out to evaluate the transfer of excess pore pressure to a proposed initiation area for the Storegga slide, approximately 150 km downslope from the Ormen Lange gas field. The results of the simulations showed development of high excess pore pressure that spread laterally towards the Storegga slide area (Kvalstad et al. 2005). In an area with low deposition rate, as in the distal part of the Storegga slide, the transferred pore pressure can be higher than the increase in overburden stress and result in swelling and unloading. The swelling will introduce a delay in peak pore pressure response. This effect was demonstrated numerically with finite element analyses and shows that the most critical conditions may develop with a considerable time lag compared with the deposition history and offset the pore pressure build up in the Storegga slide into the early Holocene.

Earthquakes

The glacio-isostatic rebound following the deglaciation of Scandinavia was responsible for strong earthquakes and evidence of magnitude 7+ earthquakes is found from onshore faults in northern Norway (NORSAR 2002; Solheim et al. 2005). Sea level curves show that the rate of uplift changed from fast and exponential to slow and linear about 8000 years ago. It is reasonable to connect the period with strong earthquakes with fast uplift and the triggering of the Storegga slide. Recent modelling shows that the seismic energy can induce ground shaking that lasts longer than previously assumed in the deep sedimentary Møre basin (NORSAR 2002). In the Storegga region, the isostatic deformation and reactivation of Late Jurassic-Early Cretaceous faults because of sediment loading on the North Sea Fan probably caused an elevated earthquake level. At present there is still higher seismic activity around the Quaternary depocentres than elsewhere along the Norwegian continental margin. A magnitude of 5.4 earthquake was recorded in the distal part of the North Sea Fan as late as 1988 (NORSAR 2002).
Figure 14: Cross section of the lower part of the Storegga slide. The glacial sediment load in the upper part of the North Sea Fan caused build-up of excess pore pressure, which may have been transferred to lower parts of the Storegga slide area, where the overburden was thin and locally steeper slopes existed.

Gas Hydrate Melting
Gas hydrates are ice-like crystals of gas molecules encaged in an ice crystal structure of water molecules. Gas hydrates are stable above a certain pressure and below a certain temperature. Methane, CH$_4$, is the most common natural gas in sediments and thus the most common hydrate former. Typically methane hydrate will form in sediments below 300 to 500 m water depth. The base of the hydrate stability zone depends on the geothermal gradient.

At the base of the gas hydrate stability zone (GHSZ), free gas may form when methane hydrate dissociates. As the depth of the base of the GHSZ depends on the pore water pressure and temperature, the base of the GHSZ can be seen as a reflector following the variations of the seabed, but diving deeper with increasing water depth. This reflector is called a Bottom Simulating Reflector (BSR) and can be an indicator of gas hydrate in the sediments above the BSR. However, a BSR is not a proof that gas hydrate exists. Gas hydrates in sufficient concentration may partly cement the sediments, and this may affect the P-wave and S-wave velocity. Attempts have been made to quantify the gas hydrate saturation level based on observed seismic velocity changes. However the uncertainty in these methods is considerable and only direct measurement using special sampling techniques can confirm the presence of gas hydrates.

At the Storegga slide region, temperature changes caused by warm water influx after the last deglaciation were limited to water depths shallower than about 700 m and thus limited to the upper slide area. The potential melting front along the base of the gas hydrate stability zone does not follow the stratigraphy and the observed slip surfaces. However, gas hydrate models show good correlation to the position of the headwall in the north-eastern part of the slide (Mienert et al. 2005). Destabilisation of gas hydrates may locally have influenced the pore pressure in this area and contributed to the slope failure.

Slide Initiation Area
The combined effects of excess pore pressure, exposure of ooze, relatively steep slopes with inclination of 10 -20° and proximity to the epicentres of potential earthquakes, favour an initial slide in the distal area (Figure 14). Here the relatively thin sediments draping pre-existing scarps may have been destabilised by excess pore pressure transferred through
the Brygge oozes or by failure in the ooze sediments. The geological evaluations and geotechnical analysis and modelling do not support the initiation of the Storegga slide as a shallow slide higher upslope (Kvalstad et al. 2005).

**Slide Process and Development**

A submarine slide development like the Storegga slide involves a material transition from a solid to liquefied state. In general, the slide process is described as having 3 main phases: initial failure and formation of blocks and slabs, debris flows, and finally turbidity currents that continue to the most distal parts (Figure 12). The seabed morphology and seismic facies of the Storegga slide indicate that the main slide developed by a headwall retreating upslope with unloading at the toe as the main driver (retrogressive slide). Most of the slide sediments were transported into the Norwegian basin as gravity flows, possibly combined with hydroplaning and turbidity currents. A high mobility of slide sediments is needed to explain the sediment transport out of the slide area for the main slide phases. The development of an erosion channel in the central scar may have enabled channelized flow and increased velocity of the debris flows. In the last stages, the headwalls were modified creating slide lobes that did not move very far (lateral spreads).

The retrogressive sliding process required that the unloading of the headwall caused strain concentrations and strength loss in the base marine clay layer that acted as gliding plane (strain softening behaviour), and that failure started to propagate upslope along this layer. The less sensitive glacial clay above was exposed to expansion and accelerated into the slide scar under gravity loading and a new headwall was created. Modelling of the process shows that the slide blocks develop a high velocity (10-20 m/s) and a high pressure near the front (Kvalstad et al. 2005). The remoulded clay combined with ‘lubrication’ by water entrapment and hydroplaning may explain the high mobility and transition to a liquefied state of the slide sediments. This retrogressive failure process continued to develop upslope as long as sufficient mobility existed in the wasted mass and soil conditions were favourable for sliding. The process stopped when the headwall met the horizontally layered and overconsolidated glacial deposits on the shelf and the mobility of the slide blocks decreased. In the last phase, the slide blocks provided support to the headwall.

**The Retrogressive Slide Model**

The Storegga slide scar and the debris formations have similarities with those of a quick-clay slide. However, the sensitivity of the investigated marine clays is much less than typical quick clays. Kvalstad et al. (2005) developed a model for evaluation of retrogressive sliding. The retrogressive slide scenario is described below.

- An initial slide is developed in the lower part of a slope with sensitive clay layers.
- The mobility of the slide material is sufficient to reduce the pressure against the initially developed headwall.
- Unloading of the headwall causes undrained expansion of the soil towards the scar and strain concentrations develop in the toe area of the headwall.
- The strain concentrations cause strain-softening in the base layer and progressive failure develops.
- The factor of safety falls below unity and the failing soil mass accelerates into the existing slide scar under gravity loading, creating a new headwall. The released potential energy is partly consumed as friction along the base and circumference of the slide mass, and partly in remoulding of the slide material along the slide base and internally in the slide mass.
- The reduction in strength gives sufficient mobility to unload the next headwall and the
process repeats itself until soil strength parameters, layering or geometry change sufficiently to reduce mobility and decelerate the sliding process.

A series of finite element analyses were carried out on a 100 m high slope with 30° inclination to evaluate the effect of sensitivity and brittleness on the development of progressive failure (Kvalstad et al. 2005). A non-linear strain softening material model was applied. The slope material is normally consolidated clay resting on a sensitive clay layer over a strong base. The front headwall was unloaded by applying pre-excavation effective stress tractions on the exposed headwall. Figure 15 shows the distorted mesh for a simulation with a sensitivity, $S_t = 3$. The strain softening effect was a drop in the factor of safety from 1.37 ($S_t=1.0$) to 1.08. A further increase in sensitivity to $S_t=5$ caused the factor of safety to drop below unity. This is then the required starting point for a retrogressive failure. The mobility of the mass after initial failure controls the retrogressive development. High sensitivity results in high initial accelerations and high mobility.

Kvalstad et al. (2005) studied the dynamics of the retrogressive sliding process using both a simplified block-and-wedge model (Figure 16) and the computational fluid dynamics code CFX (AEA Technology 1999). The details of these simulations are provided in Gauer et al. (2005). Figure 17 shows a snapshot from simulation of slide development and run-out development of a sensitive clay layer with an initially steep slope. The initial strength as well as the average remoulded strength were reduced considerably compared with today’s strength in the headwall area to allow initiation of a failure process. Variation of strength parameters gives variable results regarding run-out and failure type from slumping with limited deformation through partial run-out with deposition of separated masses to complete remoulding and debris flows with large run-out distances. The parametric studies showed that, with today’s strength (intact and remoulded), a run-out process cannot be started from the upper headwall. If a failure should be generated locally, the analyses as well as the bathymetric information indicate slumping with very limited displacements.

![Figure 15: Progressive upslope spreading of shear bands in a strain softening base layer and in the slope material due to unloaded in the toe area. Deformed mesh plot showing degree of strength mobilisation in colours (Andresen & Jostad 2007)](image)

The combined analyses of the slide scar morphology, seismic stratigraphy and seismic facies have enabled a possible sequence of events to be established for the evolution of the Storegga Slide:

1. A strong earthquake triggered an initial failure in the relatively steep slopes that existed in ooze of the Brygge formation in the distal part of the Storegga slide area (Figure 14). The pre-slide slope inclination as well the pore pressure was most likely to be higher in
this distal area than elsewhere in the Storegga area at that time.

2. The initial slide in the distal headwall removed the toe support and the shear strain increased to a critical level in strain concentration zones. As a result, progressive failure developed in the deeper sediment layers (S2) in the central sector. The rate of failure progress depends on the depth of the glide planes with shallow sliding retrogressing more quickly than the deeper ones. The effect of this difference is observed in morphological analysis that shows the main part of the northern slope failed in the sub-unit O3 before the deep failures in central upper slope area.

3. When slide reached the Ormen Lange area the headwall height increased to about 500 m with the break up of the Saalian glacial fan. The volume and energy released by the deep failures in this area produced the tsunami and generated compression and shear zones at the western flank. The shearing was followed by extension along the southern R Headwall and the southern sector failed in a south to north direction.

4. The last stages of the slide development include final shaping and filling of the debris flow channels in a southeast to northwest direction and development of the embayments in the central sector head scarp.

Figure 16: Multi wedge-block model for dynamic simulation of retrogressive slides allows estimation of run-out and cut back distances

Figure 17: Computational fluid dynamics simulation of retrogressive slide process, towards upper headwall of the Storegga slide compared with seismic profile

POST STOREGGA SLIDE SITUATION
After the Storegga slide, the headwalls must have been in a condition very close to failure and, as can be seen from Figures 2 and 5 all stages of post failure conditions existed. In the continuation of the slide base layers, under the headwall toe area, considerable softening had developed due to undrained expansion and strain development. The contractive (sensitive) marine clay materials in the shear bands had high pore pressure relative to the surrounding soil and the pore pressure started to dissipate. This applied also to the slide base underneath deposited debris.
The soils under the slide scar and behind the headwall were unloaded and this created immediately a corresponding under pressure in the pore water. This started a swelling process and in the following process of pore pressure equalisation towards a stationary condition, fluid was gradually transported into the soils due to flow from seabed and from the shear bands with high excess pore pressure. The shear band zones were thin and would more quickly equalise with the pore pressure in the surrounding soil and thus approaching the mean effective stress situation prior to the slide. The equalisation of pore pressures in the complete soil volume is a longer-term swelling process. It is indicated by calculations that today the equalisation process is finished or close to a stable stationary condition.

As the swelling continued, associated volumetric and shear strains developed with concentration of shear strains in the headwall toe areas. The effective stress reduced gradually towards a stationary condition and the material in the slide scar and in the headwall areas approached today’s overconsolidated state with reduced undrained shear strength compared to the time of the slide. The swelling-induced shear strain concentrations in the toe area might have generated slickensides in the continuation of the failure surface.

The undrained deformation and the following drained swelling process have released most of the elastic energy in the headwall front area. A restart of a retrogressive failure requires renewed development of strain concentrations. The only trigger with energy of concern for the present conditions is earthquake loading, which is shown to only cause limited strain accumulation.

CONCLUSIONS

The Storegga slide can be described and modelled as a retrogressive slide. The retrogressive model is compatible with the overall morphology in the upper slide scar and the observed features can be simulated and explained with numerical models and realistic soil parameters for the conditions at the time of the slide. The location of the initiation area and the triggering mechanism of the initial phase of the slide have not been determined with certainty. A major earthquake, toe erosion in the deep part of the slope, diapiric and water expulsion processes in the ooze layers, which had a limited coverage of marine deposits in the lower part of the slope, could all have contributed. The Storegga slide has effectively removed the evidence of such events, but work is still ongoing to improve the understanding of the conditions prevailing in the lower part of the slope prior to the slide.

The slope instability was most likely created by excess pore pressure that reduced the effective strength of the marine clays. Excess pore pressure may build up when fine-grained, water-rich marine clays are rapidly loaded by glacial debris flow deposits. The pressure build-up depends on permeability, which also controls transfer of excess pore pressure to potentially more unstable areas, e.g. with less overburden and steeper slopes. Sliding is unlikely to occur unless an external trigger is applied. Earthquake is the most acknowledged triggering mechanism, which was at its highest in Scandinavia between 10ka and 7ka as a response to glacial-isostatic rebound. Recent modelling of earthquakes generated from faults in the area show that the time of ground shaking can be up to 1-2 minutes due to the effect of the deep Møre sedimentary basin. The seismic offshore activity seems to cluster in and around Plio-/Pleistocene depocentres, such as the North Sea Fan.

The combination of locally steeper slopes, near-exposure of pre-glacial ooze, and distance to earthquake centres favours an initiation of the slide on the lower slope (Figure 3). The initial
Headwall-unloading may have occurred on the mid- to lower slope due to excess pore pressure transferred through the Eocene to early Pliocene ooze deposits created by the loading of the North Sea Fan (i.e. lateral pore-pressure transfer due to differential loading). The unloading may also be connected to crater-formation within the ooze deposits and subsequent ooze-diapirism.

Further slide development involved several slide processes, as seen from the complicated seabed morphology within the slide scar, including cross-cutting and overlying slide lobes, shear zones and slide impact compression zones. However, the overall slide development is interpreted and modelled to happen retrogressively (i.e. upslope headwall retreat), and coming to a final halt when the flat-lying over-consolidated glacial shelf deposits were reached. The uppermost slide blocks are only slightly rotated and show minor lateral movement. In fact the blocks form a retaining wall, thus adding to the stability.

The sensitivity of the marine clays is higher than for the glacial clay at the same consolidation stress and makes these layers the preferred lateral slip planes. The present-day headwall consists of stable over-consolidated glacial clays. A prolonged period of marine deposition, followed by a new glacial advance to the shelf edge and rapid deposition of glacial clays, is required in order to create a new unstable situation in the area.

REFERENCES
Relationships on the North Atlantic Margin, B. T. G. Wandaas et al. (Editors), NPF Special Publication 12, 207-239.


ACKNOWLEDGEMENTS
The authors gratefully acknowledge the work of their colleagues and peers whose contributions only received a passing mention in the paper.

484
LANDMARK LANDSLIDES IN MALAYSIA

Mohd Asbi Othman

Mohd Asbi & Associates, Malaysia

Mahadzer Mahmud

Kumpulan IKRAM Sdn. Bhd., Selangor, Malaysia

Ashaari Mohamad and Mohd Jamal Sulaiman

Slope Engineering Branch, Public Works Department, Kuala Lumpur, Malaysia

Abstract: In Malaysia, being a tropical country, landslides are often associated with intense and prolonged rainfall. This paper describes four landmark landslides in Malaysia, i.e. the Highland Towers collapse, debris flow at the tunnel bypass Road, Genting Sempah, rockfall at Bukit Lanjan and Taman Hillview landslide.

INTRODUCTION

Malaysia, being a tropical country, experiences wet tropical climate with an average annual rainfall exceeding 3500mm, which explains that the majority of landslides are triggered by prolonged and intense rainfall. Rocks are weathered at great depth as a result of warm and wet climatic conditions. Road and highway constructions involving deep cuttings into these residual soils are quite common in this country.

Most reported cases of landslides in residual soil/rock are on man-made slopes. The presence of relict discontinuities in open cuts is often ignored in the design of such slopes. Although the decomposition from weathering is intense, relict discontinuities inherited from parent rock are commonly well preserved. These relict discontinuities, which form planes of weakness, can easily be seen in many road cuttings. Such landslides are common in weathered volcanic and metasedimentary rock slopes in the northeastern part of West Malaysia (Irfan & Woods 1988). Landslides in granitic residual soil have also been reported elsewhere (O'Rorke 1972) and along Kuala Lumpur-Karak Highway (Othman 1989).

Landslides in man-made slopes are often triggered by intense and prolonged rainfall coupled with faulty or/and insufficient drainage system provided. Since the material in man-made slopes is usually homogeneous, the failure is rotational type, often circular in depth and plan.

Debris avalanche/flow and earth flow are usually associated with very intense rainfall (Morgenstern & de Matos 1975). One of the most catastrophic reported incidences of debris flow occurred in southern Thailand in November 1988 in which several villages along the rims of Khao Luang Mountains were wiped out (Phien-Wej et al. 1993). The incident claimed 373 lives and US$ 280 million of property damage were reported. Extremely intense rainfall in the order of one third of the whole mean annual rainfall was experienced in the whole of southern Thailand in just 48 hours.

This paper describes four landmark landslides in Malaysia, i.e.

1. the Highland Towers collapse
2. rock fall at Bukit Lanjan
3. debris flow at the tunnel bypass Road, Genting Sempah
4. Taman Hillview landslide

THE COLLAPSE OF 14-STOREY, BLOCK 1, HIGHLAND TOWERS, HULU KLANG, SELANGOR

Introduction
The collapse of one of the three blocks of Highland Towers Condominium, Hulu Kelang, Selangor, Malaysia at 1.30 pm on Saturday 11th December 1993 killed 48 people. The collapse of the 14-storey Block 1 was triggered by a slope failure behind the tower which weakened the foundation of the structure (see Figure 1).

Figure 1: Aerial photo of the collapsed highland tower condominium (inset photo – closer view of the collapsed block 1)

Following the Highland Tower incident, the Government, through the Local Authorities, decided to stop all ongoing hill slope developments, review all new hill slope developments which had been approved but had not started and temporarily freeze all hill slope developments seeking approval. Later, developers, who wished to resume works or seek approval for new hill slope developments, must obtain an independent specialist consultant’s report satisfying all requirements outlined by relevant authorities.

In order to explain the event effectively, it is important to establish the mode and sequence of both the slope and building failures and, more importantly, how they are interrelated and linked.
Background
The Highland Towers Condominium is located east of Kuala Lumpur city in the district of Hulu Kelang, Selangor (see Figure 2). It is one of the earliest luxury apartments built around the city during the period of 1974 to 1983. The three towers were constructed in phases; Block 1 was completed and occupied in 1979.

The Highland Towers Condominium is located at the foot of a hill slope in Hulu Kelang. Vertical aerial photographs taken in 1985 revealed extensive earthworks were undertaken behind the blocks presumably for further development of the hill slope. The slopes behind the blocks were terraced and protected by a series of rubble stone pitching walls of differing height and thickness. The maximum height and thickness of the rubble pitching walls were 7 metres and 1.3 metres respectively. Altogether, there were thirteen rubble walls built on the hill slope reaching some 100 metres above the foot of the slope (see Figure 3).

Each block consisted of two identical towers connected by a staircase and lift lobby. Each tower, in turn, consisted of a lower open ground floor car park with no wall infill, a ground floor, eleven floors of luxury apartments and a penthouse located on the topmost floor. The structures were built using reinforced concrete and founded on used rail piles driven into fill and weathered granite.

Extent of Slope Failure
The overall length and width of the area affected by the slope failure were approximately 120 metres and 90 metres respectively (see Figure 4). The centerline of the slide along the length of the affected area was nearly perpendicular to the north direction, passing close to the southern edge of Block 1. It was estimated that the slide involved some 40,000 cubic metres of earth and four slip headwalls were observed during the aerial inspection carried out two days after the tragedy, indicating that there had been four or more retrogressive slips (see Figures 4 and 5). The slips lasted approximately three to five minutes before initiating further movement down slope affecting the foundation and structure of Block 1.

Tension cracks were observed around the crown and perimeter of the landslide; the most affected area being the southern perimeter (see geomorphological map in Figure 6). The terminology used to describe the landslip follows that suggested by Varnes (1978).

Site Investigation
In order to establish the cause of the collapse, the Local Authority (Ampang Jaya Municipal Council or MPAJ) carried out extensive site investigation comprising desk study and surface and subsurface investigations.

Desk Study
In the desk study, available information relevant to the Highland Towers Condominium site was thoroughly studied. This included a detailed examination of the earthworks and engineering drawings. No records on maintenance, as-built construction and drainage records were available.

Aerial photographs available from as early as 1966 to 1993 were studied to capture relevant information related to the development progress of the hill slope and surrounding areas such as land and vegetation clearings. The progress of surface erosion and earth movement, hydrological changes and human activities were also studied from these photographs.
Topographical and geological maps together with other geological and geomorphological information were gathered to provide useful input to the investigation.

Rainfall records from four rainfall stations located within the vicinity of the Highland Towers were also examined. Data from as early as 1976 was studied to obtain information on the monthly rainfall total and duration, and annual maximum rainfall intensity. Records showed that several rainstorms which occurred just prior to the collapse were not extraordinarily high; they were typical monsoon rainstorms.

**Surface Investigation**

Surface investigation carried out comprised land survey, surface geological and geomorphological mapping, trial pits, in-situ density and surface infiltration tests, dye and isotope tracings, foundation and material inspection, etc.

The unaffected ground and rubble walls behind the adjacent Blocks 2 and 3 were continuously monitored even after the rescue operation was completed. However, lateral movements recorded over a period of two months after the collapse were negligible and within the accuracy of the survey.

Surface tension infiltrometer tests (Topp & Zebchuk 1985) carried out around the slip area, using constant zero head indicated that the material is quite porous (see Othman 1989); saturated permeability at the ground surface was in the order of $1 \times 10^{-4}$ to $1 \times 10^{-5}$ m/s.

The remains of the foundation and superstructure of Block 1 were inspected for a better understanding of the collapse mode and sequence. It has to be stressed here that the observation made must be interpreted carefully since the remains were mostly due to the impact of the collapse and not those that explain the behavior of the structure immediately before collapse.

**Subsurface Investigation**

After a close examination and analysis of the available data obtained from the desk study and surface investigation, the scope of subsurface investigation was outlined. Investigation boreholes were carried out using foam rotary drilling where soil samples were taken with minimal disturbance. These samples were closely examined to establish the fill and in-situ soil interface, and the subsurface weathering profiles. Figure 5 shows a typical cross-section behind Block 1 before failure, the extent of fill and in-situ material.

Constant head permeability tests were also conducted in these boreholes. Saturated permeability obtained was between $1 \times 10^{-7}$ to $1 \times 10^{-5}$ m/s, which was comparable to those obtained by Othman (1989) in similar material.

Standpipe piezometers and inclinometers were also installed near the failure site and other areas within the Highland Towers slopes to monitor the hydrostatic groundwater table and any subsurface movement respectively.
Figure 2: Location plan

Figure 3: Ground profile before failure
Figure 4: The affected area

Figure 5: Retrogressive slip/subsurface profile
SITE GEOLOGY, HYDROLOGY AND GEOMORPHOLOGY
The Highland Towers Condominium site is located within the Granite Formation of Mesozoic or younger era (Tan & Komoo 1990) – see Figure 7. Boreholes carried out confirmed that the hill slope consists of 3 – 6 metres of residual soils to highly decomposed granite of Grades IV, V & VI underlain by moderately decomposed to fresh granite (Grades I, II & III). Above this granite formation, fill material of approximately 4 to 7 metres was encountered (see Figure 8). Surrounding areas consist of schist and limestone.

From the aerial photos, it is evident that the contributing catchment area behind the towers is large reaching some 70 acres; the ridge is approximately one kilometre in distance from the foot of the hill slope. It is also evident that a portion of the land developed near the ridge is within this catchment.

Clearing of land and vegetation within the catchment for further development, to some extent, changed the hydrological condition of the hill slope. Within the landslip area, several locations of water seepage were observed. The broken rubble walls exposed trapped earth indicating a long period of water seepage via cracks and voids in the walls.

To the south of the landslip, a large scar of almost similar in size was observed. With the aid of aerial photographs, it can be seen that the scar which started from a small gully in 1966 grew larger over the years; the exact date of the eventual failure is not known.
Findings
The investigation focused on the events before the eventual collapse; what happened thereafter resulted in ruins which must be treated as the impact or aftermath of the collapse.

It was also important to be able to explain the manner in which the building collapsed, i.e. the toppling is slightly skewed (see Figure 4). To explain this, the mode and sequence of the slope failure must be linked to that of the structural failure.

It is widely accepted that water is the main triggering agent to this slope failure. Judging from the relatively gentle slope profile, it was most likely that the failure was due to high positive pore water pressure which developed when the rate of inflow of either surface or subsurface or both exceeded the transmissivity of the soil mass. As a consequence, the material comprising mostly loose fill lost its strength and translated under gravity. It was suspected that the majority of the water came from surface water flowing from the upper catchment where land clearing and earthworks for development were extensive. The situation was exacerbated by soil creep of the loose fill which occurred over the years resulting in large tension cracks of more than 0.5 metre in width. The presence of these old tension cracks were quite evident from the geomorphological mapping carried out after the slope failure.

Figure 7: Plan of geological formation of Hulu Klang area

The first slip involving mainly fill material could have started at the lower portion of the slope where the external covered car park and badminton court were located. This explains the witnesses’ account of the car alarm triggered five minutes before the building collapse (see
sequence of collapse shown in Figures 10 to 14). Consequently, the upper portion of the slope started to move. From the aerial inspection made 2 days after the collapse, four slip headwalls could be seen, thus confirming the retrogressive nature of the slips (see Figure 4). Figure 10 shows the large tension cracks and soil heave at the external car park during the early stages of the slope failure indicating deep seated nature of the slide. Site inspection revealed that insignificant amount of the failed soil mass actually made any contact with the structure.

These slips affected up to the eighth level rubble wall situated some 90 metres vertically above the ground floor level of Block 1 (see Figure 9). Such massive landslide rocked the building when the foundation (driven spot welded used rail piles) was subjected to the lateral force created by the slide. The situation worsened when the rubble wall in front of Block 1 failed under the force which reduced the lateral support of the foundation.

The piles at the front of Block 1 along the southern edge of the building started to get dislodged from their pile caps resulting in the explosive failure of the front columns, the weakest part of the building (no infill walls). This initial explosion of the column occurred at the front of the building. The initial failure of the front row columns also explains the manner in which the building collapsed, i.e. forward toppling.

Since the centreline along the length of the landslide almost coincided with the southern edge of the building, the effect on the foundation was worst on this side, hence the tendency to twist the structure. This explains the reason for the building to topple in a slightly skewed manner.

Figure 8: Cross-section of geological formation of Hulu Klang area
Figure 9: Cross-sections of the slope before and after the failure

Figure 10: Collapse of Rubble Wall and Soil Heave behind Block 1
Figure 11: Rubble Wall South of Block 1 Started to Fail

Figure 12: Further Collapse of the southern Rubble Wall
As population gets larger in the Klang Valley (Kuala Lumpur City and its surrounding areas), the need for high-rise condominiums and apartments is also growing. Scarcity of suitable land is slowly pushing development to difficult ground and terrain. Steep hill slopes and ex-mining
land are no longer spared for development to meet public demand for more offices and residential apartments.

Painful lessons are learnt from the Highland Towers tragedy. It is hoped that future development places great emphasis on hydrogeology and geotechnics to restore public confidence on high-rise structures on hill slopes in Malaysia. Existing tall structures on hill slopes should be regularly inspected and monitored, and necessary precautions and measures should be taken to ensure no similar recurrence of Highland Towers tragedy.

**ROCKFALL AT BUKIT LANJAN**

On the 26 November 2003 at about 07:16 a very large rock slope failure occurred at kilometre 21.8 of the Bukit Lanjan Interchange on the New Klang Valley Expressway (NKVE) (see Figure 15).

![Figure 15: Location map of Bukit Lanjan rock slope failure](image)

The rock slope failure involved an estimated 35,000 cubic metres of rock debris, mainly angular blocks of various sizes, which came to rest on the expressway (MAA 2004). The failure materials blocked the entire expressway forcing the closure of the road to the public (see Figure 16).

![Figure 16: Rock slope failure at Bukit Lanjan Interchange on 26 November 2003](image)
The failure at Bukit Lanjan occurred on the steep cut slope on the southern end of an approximately north-south trending cut. The failed slope incorporated six benches reaching to a height in excess of 65 m from the road level. The failure surface was shaped like a wedge, where the northern margin of the failure exposed a continuous major discontinuity (most likely fault plane), while the southern end was obscured by rock debris. Individual rock blocks within the debris had an estimated volume of up to about 40 $m^3$. The crest of the failure had a stepped, almost vertical face, apparently controlled by another major discontinuity.

**Geology**

The large-scale rock slope failure site in Bukit Lanjan is underlain by the granite bedrock, one of the most common igneous rock masses associated with the formation of the Main Range. Rock samples from failure material have been used to determine the specific rock type. The general texture indicates that the constituent mineral grains occur in approximately equal sizes. Major mineral contents are quartz (40%), alkali feldspar (30%) and plagioclase feldspar (25%), with minor mineral contents muscovite (4%), biotite (1%) and iron oxide (trace). Based on constituent minerals and their average grain size, the rock material can be classified as medium-grained muscovite granite. The bulk of rock debris that failed was generally fresh (Grade I) to slightly weathered (Grade II) rock materials. These materials by its own nature are competent and very strong in terms of their uniaxial compressive strength, therefore, very unlikely the cause of slope failure.

Rock masses usually inherit extensive planes of weakness (discontinuities) that result in the rock slope consisting of blocks of rock materials of various sizes. The type of discontinuities and their orientations determine the general shape of the individual blocks and usually play an important role in controlling rock slope stability.

There are generally two types of discontinuities existing in rock mass, namely, major and minor discontinuities. Major discontinuities include fault planes, shear zones, dykes or major joints. They are generally continuous over some distance (i.e. more that 50m in length) and usually can be traced from aerial photograph or observed at the entire slope face. Minor discontinuities, on the other hand, are several joint sets or fractures, usually short (i.e. less than 10m in length) and intermittent.

The rock mass in Bukit Lanjan consists of several major discontinuities. Based on field measurement, there are at least three major discontinuities, namely, Fault Plane F1 (dip/dip direction: 80°/225°), Fault Plane F2 (dip/dip direction: 78°/327°) and major Joint Sets J1 (dip/dip direction: 60°/070°). These are important geological structures seen as major factors contributing to the large-scale rock slope failure.

**Analysis of Failure**

The stability of the rock slope in Bukit Lanjan is controlled by the characteristics and orientations of the discontinuity planes present in the rock mass. Depending on the orientations of the relevant discontinuities, rock slopes could fail in these forms: planar, wedge, toppling or the combination of these three modes.

A kinematic rock slope stability analysis was conducted using a stereographic projection technique. This method is extremely useful, particularly to project three dimensional discontinuity planes onto two-dimensional presentation. Three measured major discontinuity planes were plotted on a Schmidt lower hemisphere equal angle stereonet together with the orientation of the failed slope face. The result of the stereonet analysis is shown in Figure 17.
The analysis indicates that the most likely mode of failure is wedge failure which occurred on the intersection of two major discontinuities, namely Fault Plane F2 (dip/dip direction: 78°/327°) and major Joint Sets J1 (dip/dip direction: 60°/070°). The line of intersection plunged at an angle of 33° with the direction of sliding toward 140°. Fault Plane F1 (dip/dip direction: 80°/225°) may be acting as a release plane. The development of groundwater pressure within this release plane could have reduced the resisting force, subsequently promoting failure.

Due to the favorable orientation of major Joint Sets J1 (dip/dip direction: 60°/070°), the slope may also have failed as planar failure along this J1 plane, with the direction of sliding toward 70°. Under this condition both faults F1 and F2 could have acted as release planes. However, the actual mode of failure may not have been simply wedge or planar, but could also be a combination of both types of failures.

Back analysis to assess the stability condition of the wedge was conducted using the simplified method recommended by Hoek & Bray (1981). The wedge failure was modeled for two cases: dry condition where the wedge was supported by shearing resistance alone; and wet condition where the wedge was affected by groundwater pressure and resisted by shearing resistance and cohesion of the rock mass. Based on the analysis, the calculated factors of safety (F.O.S.) against wedge failure under dry condition was found to be 3.12, and 0.41 under wet condition. Based on this result, it can be deduced that the failure was likely to have occurred under wet and high water pressure conditions.

Cause of Failure
Rock slope failure can occur when there exist several causal factors with a single or multiple triggering factors. More often than not, the triggering factor is either man-made or natural events that cannot be controlled or avoided. In the case of Bukit Lanjan, the analysis shows that unusual prolonged rainfall prior to the failure event may have been the triggering factor for the rock slope failure.

Based on the analysis of failure as discussed above, it appears that one of the major factors causing rock slope failure was rock mass structure, i.e. an unfavorable discontinuity orientation toward failure. The rock slope failure at Bukit Lanjan was due to a huge wedge block which was formed along the intersection between a steeply dipping Fault Plane F1 (dip/dip direction: 80°/225°), and a more gently dipping major Joint Plane J1 (dip/dip direction: 60°/70°), with a Fault Plane F2 (dip/dip direction: 78°/327°) as a release plane.
Apart from unfavorable discontinuity orientations, back analysis shows that failure of the wedge would not have occurred under dry conditions (F.O.S. = 3.12). However, when analyzed under hydrostatic pressure in which the faults and joints were filled with water, the F.O.S. fell below unity, indicating that failure would be expected. Under this situation, high water pressure condition is an additional factor causing rock slope failure.

Since the analysis indicated that unfavorable discontinuity orientations and built-up water pressure along these discontinuities were two most importance causal factors, one might still ask why did the failure occur several years after the construction of the slope. A more straightforward answer to this question is related to the unusual condition for the triggering factor. Rainfall analysis based on the data collected by the Malaysian Meteorological Service at Petaling Jaya and Old Subang Airport indicated that rainfall in November 2003 for both stations was extremely high. In fact, the monthly rainfall of 472mm at the Old Subang Airport station was the highest monthly rainfall since 1966.

In any case, the actual causal factors could be more complex than what was deduced, i.e. unfavorable discontinuity orientations and hydrostatic pressure. One of the most usually important casual factors, particularly in the wet tropical terrain, is deterioration of rock mass properties or the weakening of the rock mass strength through time. A significant strength reduction due to weathering of the exposed rock masses could occur over several months to several years, well within the time frame of any engineering design. Other causal factors include: characteristic of discontinuity planes (smoothness, waviness, infilling materials, etc.).
time dependent behavior (deformation, creep, displacement, dislocation, etc.) and external factors (i.e. vibration, human interference, etc.)

**Rehabilitation Scheme**

There were several rehabilitation options available that could be considered against the background of technical and construction issues, public perception, environmental consideration, and safety and operational issues. The rehabilitation scheme chosen was rock slope re-profiling because the construction was quick, simple and most cost effective compared to other options.

Rock slope re-profiling required re-profiling the rock slopes to a gentler angle. The safe overall slope angle was determined based on detailed geological and discontinuity mapping, and subsurface investigations. Based on the analysis of available data, an overall rock slope profile of 48° was considered appropriate for the anticipated in situ instability condition.

Slope re-profiling usually reduces the possibility of large scale wedge and/or planar type failure. To contain localized minor rock and debris falls, the design would have to incorporate appropriately designed catch berms, protection screens, rock trap ditches and fencing. Localized stabilization measures such as rock bolts, rock anchors, dowels, shotcrete and concrete buttress were also anticipated to support kinematically unstable blocks that would become exposed during the construction works. Drainage works were also incorporated in the form of surface and horizontal drains; the latter were designed in excess of 20 metres to 30 metres in depth. The scenery of the slope re-profiling works is presented in Figure 18.

![Figure 18: Rock slope re-profiling in progress](image)

**DEBRIS FLOW AT THE TUNNEL BYPASS ROAD, GENTING SEMPAH**

Slope failures in rural roads and highways are understandably more common due to lower factor of safety and design standard adopted as compared to that used for slopes located near
structures where higher risk is expected. However, in Malaysia, slope failures involving roads and highways seldom resulted in loss of life until the debris flow tragedy at the tunnel bypass road to the Genting Highlands. The incident which took 21 lives (one body missing) and injured 22 persons occurred at about 5.20 p.m. on the 30th June 1995.

Debris flow is usually associated with very intense rainfall (Morgenstern & de Matos, 1975). In the Genting Highlands tragedy, the tragic site is located in Genting Sempah at a curved turning at the tunnel bypass road which is approximately 425 metres from its junction at the Kuala Lumpur-Karak Highway (see Figure 19). A very intense and heavy rainfall occurred over the Genting Sempah area covering the debris flow catchment from about 3.40 p.m. to about 5.30 p.m. on the 30th June 1995 (Othman 1996). As a consequence, flash floods were reported at the highway tunnel (see Figure 19), and the surrounding Genting Sempah area resulting in a massive traffic jam in the area.

The debris flow, consisting essentially of large volume of water carrying mud, stone, cobbles, boulders and tree trunks, flowed down a stream course at great velocity and force sweeping across the tunnel bypass road at the curved turning where heavy traffic was lined up. Most of the victims were buried under tonnes of mud, some 200 metres from the point where the debris flow crossed the bypass road. It left scars in the virgin forest reserve as far as 600 metres and 265 metres in horizontal and vertical distances respectively from the tunnel bypass road. After the tragedy, the tunnel bypass road remained intact and did not appear to show any signs of distress.

**Topography and Geology**

Topographically, the Genting Sempah area which encompasses the debris flow catchment is mountainous and is dissected by fast flowing streams confined to steep V-shaped valleys. Examination of a series of aerial photographs dating back to 1966 indicated that the debris flow catchment and its immediate surrounding were never logged, developed or disturbed by human activities. The area is underlain by quartzite-shale, conglomerate and rhyolitic volcanic rocks which show localized intense shearing, faulting and jointing.

These rocks, coupled with steep topography allow the formation of thin soil cover of approximate thickness ranging from 0.5 metre to 4.0 metres, overlying debris of weathered rocks.

**Characteristics of Debris Flow**

Geomorphological mapping conducted in the area revealed that at least two natural slope failures and severe gully erosion in two other locations occurred prior to the debris flow. One of these slope failures which was in the form of a debris avalanche was triggered at the upper level within a slope hollow, some 120 metres and 75 metres in length and height respectively from the main stream. The collapse was expedited by the presence of fault zones striking across the crown of the debris avalanche. As a result of the collapse, oversteepening of the hill slope occurred on both sides of the crown of failure. This, coupled with subsequent loading down slope and an increase in pore water pressure in the process, promoted further collapse which eventually resulted in an avalanche that impounded the main stream. In addition to the witnesses' account of the incident, the evidence of impounding was also derived from mud stains on trees and vegetation near the banks of the main stream.
The earth involved in the debris avalanche was estimated to be about 5100 cubic metres. The failure mass was sufficiently large to cause a natural impounding or damming of the main stream. Other earth slumps and erosion which took place along the banks of the main stream and its tributaries did not result in similar impounding since they were relatively small in comparison to the debris avalanche.

**Rainfall Analysis**

Rainfall analysis was undertaken based on the rainfall records of Department of Irrigation and Drainage (DID) Genting Sempah rainfall recording station No. 3317004 located at about 3 km north of the tragic site. The frequency analysis of annual maximum 2-hour rainfall records indicated that the 30th June 1995 storm with a total rainfall of 96.5 mm had a probability of recurrence of about 16 years. Characterization of the storm using a total of 10 rainfall stations around the Genting Sempah area showed that the debris flow catchment was located within the centre or core of the storm.

The month of June 1995 was abnormally wet over the Genting Sempah area. A total rainfall of 428.5 mm recorded for the month of June 1995 was the highest in 20 years, approximately 2.6
times the long-term average rainfall of 163.2 mm. The 6-day accumulated rainfall ending on 30th June 1995 was recorded at 2075 mm, constituting approximately 50% of the monthly total and exceeding the June long term average of 163.2 mm by 27%. Such high antecedent rainfall which occurred within the debris flow catchment was responsible for extreme wetting of the soil prior to the 30th June storm.

The flood discharge hydrograph for the debris flow catchment indicated that storm flow would have started at 4.00 p.m. and reached its peak between 5.00 pm. and 5.15 p.m. where the discharge rate was greater than 2.5 cubic metres per second which was at least 1000 times that of the normal flow. The total volume produced by the 2-hour storm at the debris flow catchment was estimated to be about 10,600 cubic metres.

Different types and modes of slope failures were observed not only along the tunnel bypass road but also areas within a 3 kilometre radius from the tragic site. These included the Kuala Lumpur - Karak Highway, Gombak-Bentong Road and the Genting Highland Access Road. Plots of these various types of slope failures were made (see Figure 19) encompassing an area of approximately 3 kilometres in radius from its centre which coincided with the debris flow catchment.

**TAMAN HILLVIEW FAILURE (2002)**

On 20th November 2002, at about 4.30 a.m. a landslide occurred at Taman Hillview, Ampang in the State of Selangor, Malaysia destroying a double storey bungalow killing 8 people while five others were rescued from the failure site.

The collapsed bungalow is located at Taman Hillview in Ampang and details of the location with regard to previous landslides in the area such as Highland Towers and Athenaeum is shown in Figure 20.

Twelve series of aerial photographs (APs) between the Taman Hillview and Highland Towers areas of various scales ranging from 1: 3000 to 1: 25000 taken in 1974, 1975, 1985, 1992, 1994, 1995, 1996, 1997, 1999 (May and November) and 2002 were studied. The aerial photos up to the 1980’s showed that land clearing activities and development of the area of Highland Towers has just started however at the Taman Hillview area, it was still undisturbed. Streams can be seen flowing in both locations. Aerial photos in 1985 showed that filling of the valley above the failed location at Taman Hillview have been carried out and there were signs of gully erosion on the concave slope feature. In 1992, it was observed that the two disturbed surface runoff channels are still visible and minor landslides have occurred to the terraced slopes constructed in 1985 which had undergone severe erosion. The debris was then deposited in the middle of the concave slope and then transferred down to the roadside closed to the collapsed bungalow.

In 1993 and 1994, it can be observed that gully erosion had been created at the peak of the concave slope as well as the middle and the southern part of the slope. In 1997, a fresh landslide scar was observed on the upper part of the concave slope and, in 1999, severe erosions were clearly seen on the concave slope and the stream was seen flowing towards the houses. Diversion of the stream was carried out in May 1999. In 2002, the aerial photo showed the failure scar which destroyed the house. The old landslide scar identified from aerial photos in 1999 is obscured with vegetation and is just north of the present landslide. Figure 21 shows the various stages of development in the area prior to the failure in 2002.
As presented in Hussein et al. (2004), it can be summarized that the failure was due to the following:

- Previous development of the area have caused massive cuttings and dumping of loose fill material into the valley above the collapsed bungalow; and over time, this material have become soft due to heavy rainfall.
- Inadequate surface drainage to cater for surface runoff during heavy rainfall in the area and this had resulted in the ponding of water and the presence of gullies in the area.
- The total 30 days cumulative rainfall (652mm) was the heaviest since 1952 and this was also much more than the total cumulative rainfall that had caused two big landslide failures in the same area i.e. at Highland Towers (413mm) and at the Athenaeum (424mm), as shown in the Table 1.

<table>
<thead>
<tr>
<th>Year occurred</th>
<th>Location</th>
<th>30-day rainfall (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1993</td>
<td>Highland Towers</td>
<td>413</td>
</tr>
<tr>
<td>1999</td>
<td>Athenaeum</td>
<td>424</td>
</tr>
<tr>
<td>2002</td>
<td>Taman Hillview</td>
<td>675</td>
</tr>
</tbody>
</table>
Figure 21: Sample APs (1985, 1999 and 2002) from a series of twelve APs utilized in the study
REFERENCES

ACKNOWLEDGEMENTS
The authors wish to thank Public Works Department Malaysia, Majlis Perbandaran Ampang Jaya (MPAJ), PLUS Expressway Berhad and Resorts World Berhad for the permission to publish the information used in this paper. Contribution given by Tan Sri Ir. Hj. Omar Ibrahim for the Highland Towers case study is highly appreciated.
Abstract: In recent years, a number of catastrophic debris flows have revealed the high risk which features an extensive area of the Campania Region, Southern Italy. Following intense rainfall on 5 May 1998, 40 debris flows occurred on Mt Pizzo d’Alvano and hit the urban areas of Sarno, Quindici, Bracigliano and Siano. The hills and mountains of this area are covered by air-fall pyroclastic soils deposited during volcanic eruptions, which occurred in the last tens of thousands of years. In order to reduce the residual risk, a lot of studies and investigations have been carried out over the years and large and meaningful structural measures have been taken.

Besides the geo-morphological context and some relevant aspects of those events, this paper describes the main geo-mechanical and hydraulic characteristics of the pyroclastic cover and the protective works built in the Sarno area.

INTRODUCTION
On 5 May 1998, more than 100 slides were triggered on the slopes of Mt. Pizzo d’Alvano, owing to heavy and prolonged rainfall. Many slides turned into large debris flows hitting the neighbourhoods of four small towns located at the toe of the mountain: Sarno, Siano, Quindici and Bracigliano. 159 people died, the majority of whom in the Episcopio district of Sarno. Over 300 houses were destroyed or severely damaged, leaving more than 1000 people homeless. It was one of the most severe events ever to happen in Italy, which has deeply modified the way of planning risk mitigation measures and the activity of civil protection. Some laws, unapplied for years, have received a meaningful impulse and extraordinary funding has been provided. Specifically, Master Plans for landslide and flood risk mitigation have been developed all over the country to identify risk prone areas and consequent measures for risk mitigation.

After the event, the Governor of Campania was designated as the Commissioner with the aim to plan and manage structural and non-structural measures for risk mitigation in the area affected by debris flows; in addition, a technical office was set up to carry out studies, in situ investigations and to design measures for population safety. The academic staff of many Italian Universities were involved to improve the comprehension of the phenomena and set up mathematical models to support field observations. A lot of protection works have been built with a cost of over 285 million euros (Versace et al. 2007), some of which are still in progress owing to their complexity or some difficulties in their financing.

This paper describes some outstanding aspects of this complex situation with reference to the geomorphological context, the main geotechnical and hydraulic characteristics of the pyroclastic cover and the cause of the event. Finally, a special emphasis is placed on the
works built in Sarno and its surroundings to mitigate the residual risk. Information about the non-structural measures adopted in the area are reported by Picarelli et al (2007a).

GEOLOGICAL AND GEOMORPHOLOGICAL SETTING
The area concerned has an extension of around 60 km$^2$ and includes the Pizzo d’Alvano massif, a NW-SE oriented morphological structure, consisting of a sequence of limestone, dolomitic limestone and, subordinately, marly limestone dating from the Lower to Upper Cretaceous age and reaching a thickness of several hundred meters (Figure 1).

The geo-structural setting of the massif is a result of the primarily compressive poly-phase tectonic during Miocene and Pliocene, followed during the Pliopleistocene age, by trans-tensive tectonics that induced normal faulting, creating horst and graben-like structures (Del Prete et al. 1998). Starting from upper Pliocene, a marine to fluvio-lacustrine sedimentation took place in the whole area which is occupied by the Gulf of Naples and by the Sarno plain.

Owing to intense Pleistocene activity of the Phlegraean Fields and Somma Mt. volcanic centres, the alluvial deposits became intermingled with thick pyroclastic and volcanoclastic deposits, filling the graben.

The Pizzo d’Alvano relief is a broadly quadrangular and NW-SE oriented morphological structure, characterized by gently sloping summit plains, endoreic basins and relatively smooth summit ridges. Limestone strata gently dip (25°-30°) toward N-NW, forming a monocline. The limestone is affected by karstic processes which influence both the geomorphologic settings and the deep groundwater flow (Celico & Guadagno 1998).

Two joint systems, trending NE-SW and NW-SE, are the major regional structures.

Elevation ranges from 30 m a.s.l. to 1133 m a.s.l. Along the relatively steep slopes, the drainage pattern is formed by deep gullies which shape also the alluvial fans in the downhill part of the slopes (Figure 2).

The mountain slopes are mantled by very loose pyroclastic soils, produced by the explosive phases of the Somma-Vesuvius volcanic activity, as primary air-fall deposits and secondary re-worked deposits (volcanoclastic deposits).

Air-fall deposits were dispersed from N-NE to S-SE, according to prevailing wind direction, and covered a wide area reaching distances up to 50 km. Pumiceous and ashy layers, belonging to at least 5 different eruptions, have been recognized. From the oldest to the youngest, they are: Ottaviano Pumice (8000 years b.p.), Avellino Pumice (3800 years b.p.), 79 A.D. Pumice, 472 A.D. Pumice, 1631 A.D. Pumice. The average slope angle is 34°, but the maximum value exceeds 50°, whereas subvertical limestone cliffs, called “pestelle” in the local dialect, interrupt the continuity of the cover (Figure 3). The slope of the piedmont areas is, generally, less than 20°.
Figure 1: Geological map of the area with location of some major faults (from Del Prete et al. 1998)

Figure 2: Overview of Pizzo d’Alvano Mt.
The pyroclastic deposits are affected by pedogenetic processes determining paleosoil horizons during the rest phases of the volcanic activity, then the primary deposits consist of alternating layers of ash and pumice, with interbedded paleosoils (Figure 4). Secondary deposits, re-worked by sheet wash waters and by mass-wasting processes, are mainly found as debris and colluvium at the toe of the slopes in the lower part of the belt, and also in the morphological concavities present on the slopes and in the karstic depressions at the top of the limestone ridge, forming the so-called “Zero Order Basins” (ZOBs) (Guida 2003). The term “Zero Order Basins” was introduced by Tsukamoto (1973) to indicate unchannelled convergent slope located above ephemeral, intermittent or perennial first-order streams. In spite of their variability in size, shape and morphostratigraphical characters, a few of ZOB-type landforms can be identified. (Figure 5).

![Figure 3: The presence of rocky cliffs interrupting the morphological continuity of the slopes](image)

Figure 3: The presence of rocky cliffs interrupting the morphological continuity of the slopes

![Figure 4: Schematic profile of the pyroclastic deposits in the Pizzo d’Alvano area. (modified from Del Prete et al. 1998)](image)

Figure 4: Schematic profile of the pyroclastic deposits in the Pizzo d’Alvano area. (modified from Del Prete et al. 1998)
The total thickness of the pyroclastic covers ranges between a few decimetres to more than 10 m, near to the uppermost flat areas. The general structure of the soil adapts itself to the morphology of the calcareous substratum showing, therefore, complex and variable geometries.

Wide coalescing alluvial fans form the transition from alluvial plains to calcareous slopes. The considerable area and volumetric extent of the alluvial fans as well as the sedimentologic evidence suggest that, besides a consistent primary volcanic sedimentation, a great sediment supply took place from upslope, both as post-eruption remobilization of unstable air-fall volcanoclastic deposits and as debris flow activity. Karstic springs are located at the foot of the slopes. Water supply to the deep karstic aquifer is modulated by the overlying pyroclastic aquifer. At higher elevation, the bedrock is shaped by the superficial joint system, which allows the groundwater circulation and the forming of local weak springs. Deep gullies, along the Pizzo d’Alvano slopes, are scoured by ephemeral creeks that are active during intense rainfall; rill erosion is evident along the slopes. The upper flat areas and the toe of the slopes have been terraced for prevalent hazelnut cultivation. In relation to this activity, an extended network of very narrow roads and footpaths has been created. From analysis of aerial photos, a great increase in the density of these ways can be recognised in the last 30 years. A detailed analysis of geomorphological elements led to the definition of a geomorphological model referred to the Sarno slopes. With reference to the southern side of the Pizzo d’Alvano relief, the geomorphological system includes (Figure 6):

- summit tablelands;
- basin areas, divided by the morphological frame (“pestelle”) into a upper zone, where the paleo-drainage network of the limestone slopes is mainly filled with air-fall deposits or with debris colluvial material (ZOB) and a lower zone shaped by gullies, where erosional processes take place;
- area with the ancient and recent alluvial fans.
THE MAY 1998 DEBRIS FLOWS
According to the classification of flow-like landslides proposed by Hungr et al. (2001), most of the landslides that occurred in 1998, along the Mt. Pizzo d’Alvano were liquefied debris flows, even though a few flowslides were also recognized. The latter developed along gentle flat slopes characterised by relatively thin covers (Figure 7). In spite of the short width of the scarps, the involved soil mass became wider and wider during the movement toward the valley (Figure 8) giving rise to landslides with a typical triangular shape in plan.

Figure 7: Flowslide occurred in Bracigliano on 5 May 1998
In slopes cut by gullies and presenting very thick covers, the debris flow tends to get deeper and channelized.
This has been the largely prevailing typology on the Pizzo d’Alvano massif (Figure 9).

Some landslides, especially the largest ones, spread in different and successive stages, in some cases owing to the formation and the subsequent collapse of ephemeral obstructions.

The debris flows attained volumes of up to 180,000 m$^3$, owing to the contribution of different landslides occurring, more or less contemporaneously, in the same gully. A simple scheme can be roughly used to describe the mechanism of these landslides (Figure 10). In fact, relatively small soil slips and/or flowslides occurred in most of the basins of the Mt. Pizzo d’Alvano, with the triggering zones being located in the upper parts of the slopes, in the ZOB areas, near to the morphological frame, or above and below trackways (Del Prete et al. 1998).

![Diagram](image)

**Figure 10:** Scheme of a typical flowslide indicating the prevalent phenomena in the different morphological zones

The size of the landslides increased along their path downslope, digging and moving the debris which fills the gullies, while the movement pattern rapidly turned into a debris flow style. The soil masses reached the broadly urbanized piedmont areas, releasing their huge destructive power, owing to the high velocity attained during the movement (more than 10 m/sec). Figure 10 reports the scheme of most of the debris flow, based on eyewitness and accurate site surveying.

Concerning the mechanics of rupture, an univocal and shared opinion does not exit. However, the role of the rain, either fallen immediately prior to the event or accumulated during the previous wet season, is beyond dispute (Versace et al. 2003; Capparelli 2005). What appears more debatable is the identification of the phenomena that, activated by the rain, led to the development of flowslides and liquefied debris flows.
A discussion about the likely process, which led to slope failure and governed the development of flow-like movement, is reported in the next section.

**MAIN PROPERTIES OF AIR-FALL VOLCANIC ASH**

Recent catastrophic debris flows in pyroclastic soils, including those which occurred at the foot of Mt. Pizzo d'Alvano, have stimulated a number of investigations focused on the interpretation of these phenomena.

According to both geomorphological and geotechnical data supported by experience, the major risk of debris flow is concentrated in a wide zone located to the North and to the East of Naples, which is mantled by air-fall products of the Somma-Vesuvius volcanic centre. These consist of alternating layers of unsaturated ash and pumice. Because of the complete absence of interparticle bonding, such materials are completely cohesionless.

Figure 11 reports grain size curves of some deposits involved in debris flows. Despite the extent of the area concerned, these materials are highly uniform: volcanic ash displays a high sandy component and a significant amount of non-plastic silt, as well as some gravel due to the presence of pumiceous grains. The figure includes also data concerning two different outcrops (Episcopio and Lavorate) in the area of Sarno, which present a content of non-plastic fines equal to about 20%, while the percentage of pumices ranges between 10 and 20%. A higher percentage in fines is reported by Cascini et al. (2003). A high porosity, typically ranging between about 65% and 75%, is another peculiar property of these deposits. In particular, in the two sites of Episcopio and Lavorate in Sarno, the porosity falls within the range of values comprised between 66% and 72% with an average value of 69%.

Such high values, depending on the mode of formation of the deposits, reveal the presence of macropores; in addition, as suggested by Lampitiello (2004) through some photographs with SEM, sandy particles are bridged to each other through chains of silt. These observations indicate a metastable soil behaviour.

All these features, i.e. the grain size, the absence of plasticity and the open fabric, are typical features of soils which are susceptible to liquefaction.

Air-fall pyroclastic soils typically cover steep slopes. Because of slope steepness and of their relatively high hydraulic conductivity, these soils are generally not saturated. This explains why the slope angle can be much higher than the friction angle.

As a matter of fact, systematic field measurements carried out by means of fixed or portable tensiometers show that suction generally fluctuates between values close to zero, in the wet season, and several tens of kPa, in the dry season.
Figure 11: Grain size distribution of samples taken from different sites

Figure 12 reports the values of suction measured on slopes around Sarno (Figure 13). Daily average records at 0.35 m deep from the ground surface for the period May-June 2006 and November-December 2006, are plotted against time, together with the daily rainfall recorded in the same period.

Figure 12: Daily rainfall and daily average of suction measurements at 0.35 m depth, (a) during dry season, May/June 2006 and (b) wet season, November/December 2006
Because of the absence of a true cohesion, sampling is never easy. However, it can be successfully performed in the presence of suction, at least on ash, especially for sampling carried out by hand or by the help of a hydraulic jack in pits purposely excavated on the slope. In contrast, the difficulty of sampling cannot be overcome for pumices for which reliable data about hydraulic and mechanical properties are not available.

The hydraulic and mechanical properties of samples taken from several sites in the wide area mentioned above or in other sites covered by volcanic ash have been investigated by different researchers (Evangelista et al. 2002; Olivares & Picarelli 2003; Lampitiello 2004). A comprehensive review is reported by Picarelli et al. (2008).

Because of the low degree of saturation, testing must be carried out through nonconventional suction controlled procedures. In particular, the coefficient of permeability of unsaturated specimens can be obtained by the interpretation of the transient phase of suction equalisation relative to oedometer or triaxial tests on unsaturated specimens; it rapidly decreases as the degree of saturation decreases reaching values as low as 1.0E-10 m/sec for values of suction around 100 kPa. The saturated permeability can be obtained by constant head tests in the oedometer or in the triaxial apparatus; it falls within the range of values comprised between 1.0E-06 and 1.0E-07 m/sec. Values of this order of magnitude have been obtained also for the Sarno ash. Because of suction, unsaturated pyroclastic soils present an apparent cohesion. Measurement of the latter must be obtained by suction controlled triaxial tests.

An accurate procedure to do that was described by Picarelli et al. (2008) who reported data obtained on the Cervinara ash. The most relevant aspect of this experience is the strong non-linearity of the relationship between suction and cohesion (Figure 14). The latter is a few kPa for values of the suction up to 10 kPa and reaches values as high as 12 kPa for suction around 70 kPa. These numbers can explain the stability of thin covers of ash which mantles steep slopes.
Data regarding the shear strength of saturated specimens are also important to investigate the soil behaviour after prolonged rainfall. It is worth mentioning that saturation of such very loose soils provokes a volumetric collapse and a consequent change in the void ratio. This occurs also in the triaxial cell during saturation of natural soil specimens and leads to errors in the interpretation of the tests. In order to minimize these effects, Olivares and Picarelli (2001) proposed an original technique based on the use of carbon dioxide in the saturation phase.

Typically, the soil exhibits a ductile and contractive behaviour under shear in drained conditions, while its behaviour is brittle when subjected to undrained shear, reflecting a continuous increase in pore pressure (Olivares & Picarelli 2001): in other terms, as discussed above, loose volcanic ash is liquefiable.

It is well known that this behaviour depends on a number of factors such as grain size, plasticity, density, initial and induced state of stress. Lampitiello (2004) investigated the role of these factors through an extensive laboratory programme carried out on the Cervinara ash. Figure 15 reports the results of a number of undrained triaxial tests after isotropic consolidation (CIUC tests) performed on reconstituted specimens prepared at different void ratios and subjected to a confining stress comprised between 25 kPa and 159 kPa. As discussed above, the soil behaviour strongly depends on initial conditions, being contractive or dilative as a function of the initial void ratio and confining stress. However, the Steady State Line is unique, regardless of the initial state and type of test, while the mobilised strength strongly depends on the pore pressure which develops in the shear stage.

This is clearly shown in the compression plane of Figure 16. This diagram allows to separate the specimens which are susceptible to liquefaction (whose an initial void ratio is above the Steady-State Line, SSL), and specimens which are not susceptible to liquefaction (below the SSL).
Figure 15: Results of undrained compression triaxial tests on reconstituted specimens of the Cervinara ash (from Lampitiello 2004): (a) loose specimens; (b) dense specimens

Figure 16: Results of undrained triaxial compression tests on reconstituted specimens of Cervinara ash, reported in the compression plane (from Olivares et al. 2003)
Naturally, these data are very useful to select deposits which can give rise to flowslides and liquefied debris flows as a consequence of slope failure (Olivares & Picarelli 2001). Figure 15 also suggests that the peak strength can be enveloped by a line, the Instability Line (Lade & Pradel 1990), which is located below the Steady State Line.

Typically CIUC and CAUC tests (undrained triaxial tests after anisotropic consolidation) allow the identification of an Instability Line which envelopes peaks located below the SSL.

An example is reported in Figure 17 concerning specimens having a void ratio, at the end of the consolidation phase, that ranged between 1.6 and 1.7.

This suggests that a failure process which is entirely undrained can mobilise a shear strength less than the one available if rupture were entirely drained. In addition, a rupture process starting from a high initial deviator stress (CAUC tests), as typically occurs in the case of steep slopes, can favour a sudden and unexpected collapse even under a very small increase of the shear stress.

However, the experimental programme showed that the Instability Line is not unique depending on the void ratio of soil at the end of the consolidation stage; the instability domain is wider for higher void ratio. Testing on natural samples of Cervinara ash which present a void ratio well above the SSL, fully confirm these results.

Similar considerations regarding the susceptibility to liquefaction can be made for Sarno ash. Figure 18 reports the results of drained and undrained triaxial tests on natural samples taken from the Episcopio and Lavorate outcrops. The Steady-State friction angle varies between 33° and 39°, a typical range of values for air-fall volcanic ash. In spite of the ductile behaviour which characterises the soil under drained conditions, the undrained response is highly brittle, suggesting that liquefaction can really be the fundamental mechanism of flowslide generation.
The stability conditions of slopes covered by pyroclastic soils, which can reach angles well above the friction angle of the soil, is mostly due to suction which in turn depends on the degree of saturation that is normally much lower than 100%. Since the primary pyroclastic covers are generally rather thin (not more that a few meters), especially on steep slopes, a very low apparent cohesion can assure a high safety factor. As an example, Figure 14 shows that a significant cohesion is assured by a suction of very few kPa, which is available even for a degree of saturation close to 100% (Picarelli et al. 2008). For this reason, steep slopes are stable even for a very high degree of saturation.

For the same reason, since pyroclastic soils are not subjected to deterioration, i.e. a decrease of the parameters of shear strength, \(c'\) and \(\phi'\), slope failure is generally due to a decrease of the apparent cohesion \(c\), because of a decrease of suction, i.e. an increase of water content. Naturally, this occurs only for exceptional events or for sequences of closely spanned intense events.

In the Sarno area, this occurred in 1998, when monthly rainfall in April was 108.8 mm, against the mean value of 73.3 mm, and rainfall in the first six days of May was 107.6 mm, with a daily peak of 74.2 mm, against a mean value of monthly rainfall in May of 37.8 mm. As a consequence, the suction profiles in the subsoil underwent a sharp drop and possibly, ponding water formed at the base of the pyroclastic covers.

These phenomena could have been caused by two different mechanisms:

- water content increase, as a consequence of infiltration from the ground surface, which follows by a decrease of suction;
- upward water flow from the underlying fractured limestone and saturation of the lowermost layer. As a matter of fact, for many days after the event, in some zones, a continuous flow of water was observed from the uncovered limestone. Nevertheless, the occurrence of this last mechanism requires an unlikely water supply in the rock mass, directly from the ground surface, through the network of joints.

It is worth mentioning that slope failure in granular deposits can provoke different types of
landslides, such as slide, debris avalanche, flowslide, debris flow. The nature of the movement depends on the soil response in the post-failure stage (Picarelli 2000). That event mostly triggered flowslides (movement of a liquefied soil mass) and liquefied debris flows (channelized movement of a liquefied soil mass). Such phenomena can be justified only by soil liquefaction.

Liquefaction has already been suggested for landslides involving wasted coarse grained saturated materials (Bishop 1973; Dawson et al. 1998; Blight et al. 2000) and natural slopes (Sassa 2000).

However, the occurrence of liquefaction in loose saturated volcanic ash was demonstrated by Olivares & Picarelli (2001) and Lampitiello (2004) through laboratory tests. This phenomenon develops only if soil is saturated or nearly saturated and induced deformation is so rapid as to provoke a sudden increase in pore pressure (undrained condition). This was demonstrated through flume tests on initially unsaturated ash subjected to artificial rainfall (Damiano 2004; Olivares & Damiano 2007): It is discussed in detail by Picarelli et al. (2007a & b).

As a matter of fact, eye witnesses and data collected after the 1998 event indicate that the landslide evolution was characterized by spreading towards the areas located below, soil masses detached from the upper slope sector, where many failures took place, or soil deposits collapsed as a consequence of the impact of material falling from the morphological frames. In general, these soil masses channelized into pre-existing gullies, giving rise to debris flows; but in a few cases spread over flat slopes, generating flowslides. It is likely that pore pressures increased even more during movement because of the increase in the internal state of stress provoked by the loading and/or impact of the soil masses coming from the uppermost parts of the slope.

PROTECTION WORKS FOR RISK MITIGATION IN THE SARNO AREA

Structural measures designed for risk mitigation in the Sarno area have been taken following a complex and articulated strategy, with the following aims:

- to increase the safety factor of slopes in source areas;
- to restrain bed and slope erosion along gullies and then control the increase of debris flow volume along its path;
- to control rainfall water runoff toward the major rivers located in the plain (i.e. the Sarno River and Regi Lagni channel), whose beds are not adequate to contain floods even with a low return period;
- to create storage areas for debris flows;
- to channelise debris flows and lead the material towards these storage areas.

To attain these objectives, the following works have been carried out:

- bioengineering techniques aimed at reducing slope erosion;
- check dams in the upper part of the gullies, for bed erosion control and slope stabilization;
- channels for collecting rainfall water;
- retarding reservoir for flood routing;
- control structures, like sediment basins, diversion structures, transverse walls, bounding the debris flow prone slopes in order to protect the inhabited areas.
Different kinds of work have been combined in order to obtain articulated risk mitigation systems. Debris flow control has been obtained with slope stabilization, check dams, and three large deposition basins (Episcopio, Curti, Mare), located in the piedmont area above the town. Each basin is reached by more than one gully. The rainfall water drainage has been allowed by the channel network which crosses the deposition basin and reaches retarding reservoirs, located below the town, which reduce the peak flow toward the Sarno River. The Episcopio system will be described later. Some basic criteria were adopted to establish the most correct strategy.

The first problem concerns the option, perhaps only theoretical, between active works, such as bioengineering slope medications, and passive works, such as check dams, channels, basins, etc. It is not a real question, because it is impossible to entrust people’s safety only to bioengineering techniques. In fact, flowslides and debris flows are mainly triggered in areas which are characterized by slope angle higher than 30° and by a significant pyroclastic cover. This is likely in areas where ZOBs, buried channels or undrained trackways which concentrate water in impluvia, are present. Location of all the areas with similar characteristics, often sited on not easily accessible hillslopes, is difficult. Moreover, slope failure may occur also in areas where these risk factors are not so evident or are even apparently lacking.

Other aspects to be considered are: the very high cost of such works extended over all the hillslope; technical difficulties in reaching the impervious area, where the marking out of new paths could generate further landslides and change the small-scale drainage network of rainfall water; very high vulnerability of wooden works in fire risk prone areas.

On the other hand, bioengineering works are beautiful (Figure 19), they can be pleasantly inserted into the natural environment, and can be very effective if they are properly designed and located. Therefore, the right choice is the integration of both types of works: the active ones reduce the probability of slope failure in the upper parts of hillslopes, where the risk of collapse is higher; the passive ones mitigate and drastically reduce the damage induced by debris flows that could be activated despite the presence of active works.

The location of these works is another important issue. The chaotic development of urban areas has produced the loss of natural courses which debris flows followed in the past. The random growth of the towns towards the mountain and the upsetting of both the natural drainage network and the existing hydraulic works, produce a tangle of natural and man made systems. Sedimentation basins built in the past above the town are now included in this and are used for different purposes, such as a football fields.

Many channels disappear in long culverts with unknown patterns and unable to guarantee hydraulic linkage. In such cases, the main target is to separate natural and man made functions, to free water and debris flow courses and to minimize meddling with built-up areas. This goal may be achieved also by delocalization of existing houses, restoration of the natural drainage network, and a green belt boundary. In practice, this latter is unlikely, as it requires a long time, large investment, and social conflict.
When risk is pending, mitigation works are needed immediately. The better choice is to disconnect and separate the upstream natural system and the downstream artificial system, i.e. the built-up areas. So risk mitigation is quickly achieved and future land restoration is still possible and even easier.

Different works have been carried out, in Sarno, to disconnect the downstream and upstream areas. In many cases large basins with deposition areas have been adopted, which allow the debris flow to spread and slow down owing to sudden gradient decrease at breakers, discharge control orifices at basin outlet. The basin volume must be equal to the predicted volume of debris flow, as the routing effect has to be disregarded because outlet orifices occlusion is always possible during debris flow. This hypothesis, otherwise, leads to an increase of the safety factor. However, the routing effect is considered for downstream channel design, when runoff is only produced by rainfall and no debris flow is expected to occur.

More than one gully often ends in one single basin, which can collect debris flow from all of them, but also from the inter-gullies areas, so greatly increasing the safety of the downstream area. This single basin system allows the total volume for debris flow storage to be reduced by a proper reduction factor, as a simultaneous flow from all the confluent gullies has a very low probability of occurrence. The single basin system of Episcopio, in Sarno, is shown in Figure 20. The system includes slope stabilization in the mountain zone along main tracks; a large basin with a capacity of more than 170,000 cubic meters; two deep channels, with walls or levees on the outer side, which bound the debris flow prone areas. The basin was realized by digging, so its visual impact is very low. Lateral channels present a top protection to prevent avulsion in the bend.

Episcopio is the most stricken district of Sarno with 90 victims and more than 120 houses destroyed or damaged. Downstream of the basin system, many houses have been reconstructed with some extra precautions, adopting special technical rules. In fact, the ground floor cannot be used for residence and the structural frame is adequately reinforced.
Each family was free to choose between rebuilding its own house in the same place or buying a new one in another part of Sarno or in another town. In both cases, the Italian Government bore the cost.

Figure 20: Single tank system, in Episcopio

In some cases, slope morphology does not allow a single basin system to be built, so one sediment basin is needed for each gully, and more basins are present at the toe of the slope (multiple basin system). If both systems are technically valuable, the single basin system seems better, as it can capture inter-gullies debris flows and is safer in the case of debris flow produced from a single gully only.

Figures 21 and 22 show a multiple system in Quindici and details of the Connola basin. These are filled only when debris flow occurs, i.e. once in many years. During normal periods, the clear water must be diverted and cannot reach the basin, because sediments could
reduce the capacity available. Therefore, clear water must follow a different course to reach the downstream drainage system. The debris, on the contrary, has to come into the basin and only a very small part of it can flow downstream.

Then an appropriate diversion structure has to be designed, taking into account the fast flow regime. Usually this kind of basin is blind, i.e. there is no outlet, then its capacity needs to be large enough to contain all the design volume. Of course, when the basin is filled by mud, it has to be cleaned rapidly.

Figure 21: Multiple tank system in Quindici

Figure 22 shows the diversion system of the Connola basin. An orthogonal channel receives clear water up to a discharge of about 30 m$^3$/s.

For larger values, i.e. when the debris flow occurs, the flow passes over the channel and reaches the basin. The discharge in the orthogonal channel changes a little. This device may present some problems owing to both high flow velocity and conspicuous sediment transport. So diversion inside the basin seems a better choice than diversion upstream. This choice drastically reduces the risk of exceeding the designed downstream discharge. Moreover the diversion occurs in the basin with slow flow or in any case with low Froude numbers, so the risk of outflow is highly reduced. A low flow channel is built along the basin with proper outlet for clear water runoff.

If the basin length or slope, change between the upstream reach and the basin bed are not
large enough, and breakers have to be inserted into the basin; otherwise riprap protection in the inlet area suffices.

Figure 22: Detail of the Connola tank

Separation between upstream area, where debris sources are located, and downstream urbanized area, which must be protected, can also be obtained by a diversion work, which diverts water and debris flow into a not vulnerable area (Figure 23). The structure is formed by (i) a large and deep channel, with high slope, not less than 6-7%, so no debris deposition may occur inside, and (ii) a wall in the downstream side, so avulsion may be avoided.

In some cases an embankment effect is obtained by increasing the downstream channel edge, which is built higher and stronger, and can be integrated within the channel. In the Episcopio system there are both a basin and a diversion structure. In this case the latter diverts the flow into the basin area.

One of the most important works is the first upstream check dam, which withstands the highest stress. It closes the whole system upward, breaks and slows down debris flows, stabilizes both the bed and banks, traps and stores sediment transport. Moreover, it leads the flow towards the canalized reach. Open check dams, such as the one in Figure 24 can also select transported sediment, trapping the largest. When the opening is very high, these dams can work for a long time, as occlusion gradually increases upwards. Of course periodic cleaning is needed.
The breaking and slowing down effect are only transient owing to the very steep gradient which allows velocity and sediment loading capacity to increase over a short distance. Then other check dams downstream of the first one are needed. Figure 25 shows a series of check dams in the Cantariello gully in Sarno.
CONCLUSIONS
The tragedy that happened in Sarno caught our defence systems unprepared, incapable of assuring acceptable safety conditions. The event shook people’s consciences and gave new impulse to the management of soil defence and civil protection systems. A lot of resources were allocated to these aims and the situation has certainly improved, but there is still much work to do. Since Sarno and its neighbourhood was hit hard by the debris flows, numerous investigations and studies were set up to better support the operative management of the emergency. These allowed a deepening of the knowledge of these phenomena and the performance of useful simulation models. Huge funding has been devoted to the building of structural systems that have been, mostly, successfully completed, even if with some mistakes.

Nevertheless, an acceptable defence strategy from the debris flows has been conceived, which provides the correct integration of active and passive works.

As regards the containment structures, the single basin system, which can receive debris from more than one gully, greatly increases downstream area safety, as it can intercept material also from the inter-gully areas. When the slope morphology requires a multiple basin system, that is, one basin for each gully, the better choice is to allow diversion from the clear water and debris flow into the basin instead of building this diversion upstream.

The events of Sarno allowed a turning point in the management of civil protection. In fact, a control scheme has been carried out that assembles real time monitoring systems, forecasting models, and above all, the Territorial Survey composed of technical staff, engineers and geologists, who look after the territory during the emergency phases, when the models suggest the threshold values have been exceeded (Picarelli et al. 2007a).

However, even though with some contradictions, Sarno delineates a meaningful model of protective measures in Italy and indicates useful guidelines for risk mitigation control works.
REFERENCES


INSIGHTS FROM SOME MILESTONE LANDSLIDES IN HONG KONG

H. N. Wong and K. K. S. Ho
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Abstract: The acute landslide problem confronting Hong Kong is the outcome of dense urban development in a hilly terrain combined with heavy seasonal downpour and the legacy of a large number of substandard man-made slopes mostly formed before the mid-1970s with no specialist geotechnical input. As a result, serious landslides had occurred from time to time claiming human lives and causing significant socio-economic damage to the community. Following a number of disastrous landslides with multiple fatalities in the 1970s, the Hong Kong Government introduced geotechnical control and implemented a holistic Slope Safety System to combat landslide disasters. Study of significant landslides constitutes an important component of the Slope Safety System and the lessons learnt provide a basis for further enhancing landslide risk management practice. This paper gives an overview of the insights gained from a selection of significant landslides in Hong Kong and outlines their impacts on the regulatory regime and professional practice.

INTRODUCTION

Hong Kong has a population of over 7 million and a small land area of about 1,100 km², of which only about 15% is developed land. The economic expansion since the 1950s has been accompanied by a large number of man-made slopes formed under extensive civil engineering and building works, mostly with no specialist geotechnical input. The geology of Hong Kong is described by Fyfe et al. (2000) and Sewell et al. (2000). The main rock types are granite and volcanic rocks (viz. tuff and rhyolite), which predominate in the urbanized areas of Hong Kong where building development is the densest. The climate in Hong Kong is sub-tropical, with an annual rainfall averaging about 2,200 mm. Rainfall intensities can be high, with 50 mm per hour and 250 mm in 24 hours being not uncommon. The combination of hilly terrain, deep tropically weathered rock profiles, high seasonal rainfall and lack of geotechnical input has, in the past, resulted in serious landslide problems in Hong Kong’s dense urban environment (Figure 1). The scale of the problem is reflected by the fact that landslides have been responsible for the death of more than 470 people in Hong Kong since 1948 (Figure 2). On average, some 300 landslides are reported every year.

Concerted efforts have been made over the past 30 years to manage and reduce landslide risk. In the aftermath of a number of serious landslides in the 1970s, the Government established the Geotechnical Control Office (GCO) (renamed Geotechnical Engineering Office (GEO) in 1991) in 1977 to regulate geotechnical engineering and slope safety in Hong Kong. A comprehensive Slope Safety System was formulated and implemented (Chan 2007), which provides a regulatory framework and advocates state-of-the-art slope engineering and risk management practices to combat landslide threat.

The Slope Safety System has evolved with time, with significant progress made to incorporate major lessons learnt from landslide investigations. It is therefore instructive to...
review some of the milestone landslides to appreciate the role of landslide investigations in contributing to enhance the professional practice and the risk management system.

Figure 1: Dense urban development in Hong Kong mingled with man-made slopes and natural hillsides

![Dense urban development in Hong Kong](image1.jpg)

Figure 2: Known landslide fatalities in Hong Kong since the late 1940s

![Known landslide fatalities in Hong Kong](image2.png)

SELECTED MILESTONE LANDSLIDES IN HONG KONG

Landsliding can provide an invaluable source of information for advancing the understanding of the causes and mechanisms of slope failures. The findings from selected landslide investigations had resulted in major impacts on the related aspects of local geotechnical practice and led to institutional reforms of the Slope Safety System. Much has been learnt in Hong Kong about landslide risk management in the past 30 years through the study of serious slope failures.
The Po Shan Road and Sau Mau Ping Landslides in the 1970s
In 1972, rain fell almost every day from early May, becoming more intense in mid-June with a daily rainfall of over 200 mm during 16 to 18 June 1972. As a result, the two most disastrous landslides in the history of Hong Kong occurred on 18 June 1972, which brought about a total of more than 130 fatalities.

The first landslide occurred shortly after 1 p.m. in the Sau Mau Ping Resettlement Estate following a heavy downpour at about 11 a.m. with a maximum hourly rainfall of about 70 mm. The failure involved the sudden collapse of about 6,000 m$^3$ of fill material from a 40 m high road embankment inclined at 34°. The mobile debris slid down “like a carpet” and inundated a temporary housing area for more than 380 people (Hong Kong Government 1972a) and destroyed many huts (Figure 3). It resulted in 71 fatalities and 60 injuries. The road embankment that failed was completed in 1964 by depositing decomposed granitic fill in a 60 m wide valley. The fill material was largely deposited by end-tipping, which was commonly adopted in the local construction industry in the 1960s, giving rise to a loose and metastable structure due to lack of proper compaction.

Figure 3: The 18 June 1972 Sau Mau Ping landslide

About eight hours after the Sau Mau Ping disaster on 18 June 1972, a massive landslide (about 20,000 m$^3$ in volume) occurred on a steep hillside above Po Shan Road in the Mid-levels area (Figure 4), killing 67 people and injuring 20 others. A 13-storey residential block was knocked down by debris impact. The landslide was caused by the collapse of a temporary cut within a private building lot. Signs of distress, in the form of small landslides and serious settlement of the road pavement together with severe cracking, were noted at Po Shan Road two days before the landslide disaster. At about 5 p.m. of 18 June 1972, a large landslide occurred within the private building lot and the debris came down across Conduit Road, demolishing the rear wall of No. 11 Kotewall Road, a two-storey house. Eventually, the fatal landslide occurred at about 9 p.m.
Public outcry over the large number of deaths of the two tragedies led to the appointment of a Commission of Inquiry comprising a district judge, a professor in civil engineering from a local university and an architect, charged with making recommendations to avoid such disasters. A private consultant was also engaged to undertake a technical investigation of the Po Shan Road landslide for the Commission of Inquiry. The landslide at Sau Mau Ping was diagnosed as being “due primarily to softening of fill material caused by infiltration of rain-water mainly through the sloping face, as a result of an exceptionally long and intense rainstorm” and that “no fault was found with the manner in which the design and construction of the embankment was carried out” (Hong Kong Government 1972b). In addition, the post-failure forensic investigation (Hong Kong Government 1972b) established that three unfavourable factors combined to cause the Po Shan Road landslide, viz. the nature of the material forming the hillside, the exceptionally intense rainstorm, and the steep cutting in the private building site.

Some four years later, another very destructive landslide struck Sau Mau Ping again on 25 August 1976 following heavy downpour associated with Tropical Cyclone Ellen. A total of four landslides took place, all resulting from the collapse of the side-slopes of highway embankments formed of earth fill. Three of these turned into mud avalanches, the most hazardous of which was on the face of a 35 m high embankment above an occupied public housing block (Figure 5). The landslide debris (4,000 m$^3$) slid down as “a large sheet” and inundated the ground floor of the public housing block. 18 people were killed and the other 24 people were seriously injured.

Following the fatal incident, the Governor immediately convened an Independent Review Panel of six geotechnical experts, four from overseas, to report on the causes of the landslide and propose measures to prevent similar disasters in future. The technical investigation (Hong Kong Government 1977) concluded that the loose fill placed by end-tipping was liable to liquefy under intense rainfall leading to mobile debris with a long runout. The failure mechanism comprised an undrained failure involving the collapse of a metastable structure of the saturated, or near saturated, loose fill.
Observations
The potential catastrophic consequences of serious landslides to life and property were amply illustrated by the Sau Mau Ping and Po Shan Road landslide disasters. The public became acutely aware of the landslide threat. Hitherto site formation works and deep excavations were under the ambit of civil engineers, with project coordination by architects in the private sector, and the need for specialist geotechnical input was not recognised. The fundamental flaws in the manner fill embankments were constructed in those days were, however, not appreciated by the 1972 Commission of Inquiry. Due to lack of attention to soil mechanics, control on fill compaction was inadequate given the pressure to expedite the project programme with the belief that any ensuing problems, should they occur, would be minor and localized, largely of a maintenance nature. The 1976 investigation made the insightful observation regarding the generic problem associated with loose fill slopes. As a result, all man-made fill slopes formed by the same prevailing practice became suspect and potentially substandard.

At the recommendation of the 1972 Commission of Inquiry, a civil engineering section with 10 professional posts was set up in the Buildings Ordinance Office of the Public Works Department to assist the buildings surveyors and structural engineers by checking geotechnical aspects of private development submissions. Henceforth, the traditional permissible angle rules for cut slopes (Brand 1984) were no longer automatically acceptable and designs for major earthworks required justification by use of soil mechanics analysis.

The second landslide disaster at Sau Mau Ping emphasized the need for proper specialist geotechnical input in slope design and construction, given the unsatisfactory state of design and construction practice of slope works, lack of technical guidance and inadequate site supervision and enforcement of specifications at that time. The 1976 Independent Review Panel made a major recommendation on institutional reforms in the setting up of a centralized policing organization within the Government to regulate the planning, investigation, design, construction, monitoring and maintenance of slopes. This culminated in the establishment of...
the GCO in July 1977. The key challenges at that time were, inter alia, to regulate the standard of new slope formation and systematically retrofit old substandard man-made slopes. Such concerted efforts by the Government represent a landmark in slope safety practice in Hong Kong whereby empirical approaches based on precedents and rudimentary soil mechanics knowledge were replaced by more scientific approaches that represented the state-of-the-art at the time.

**The 8 May 1992 Baguio Villas Landslide**

On 8 May 1992, a localized deluge occurred, bringing more than 350 mm of rain in a 9-hour period (with a return period of about 70 years) to the western coast of the Hong Kong Island. Some 350 landslides were reported to the GEO, two of which resulted in a total of three fatalities.

One of the fatal landslides occurred at Baguio Villas, which involved the collapse of a 9 m high, unregistered masonry wall retaining a fill platform in a temporary housing area (Figure 6). Fill material (1,500 m³) was released progressively into a 100 m high, 32° gully below. Subsequent concentrated runoff from intense short-duration rainfall flushed the debris down the 170 m long drainage gully in the form of a major washout. Concentration of surface water in the gully, in combination with the large volume of debris failed, produced a highly fluidized debris mass, which struck the back of the residential block that stands at the foot of the slope, punching a hole in the podium slab and piling up against the building. The debris entered the podium-level apartment through the windows and caused the death of a 7-year old boy inside (Chan et al. 1996). A government engineer, who was on inspection duty on the podium at the time, was also killed by the landslide.

Following the landslide, a forensic investigation was carried out by the GEO (Chan et al. 1996). The investigation established that the fatal landslide at Baguio Villas was caused by the combination of the gradual removal of passive soil support in front of the wall by surface erosion and the rapid development of perched water table at the back of the retaining wall.
during the severe rainstorm. The failure of the wall removed part of the support to a large body of loose fill material and the prolonged rainfall led to the progressive failure of the unsupported fill platform. The masonry wall that failed was not included in the Catalogue of Slopes before the collapse. The major washout down the natural gully could be modelled using the FLO-2D computer program as it is akin to a wet channelized debris flow (Figure 6).

Observations

The years from 1984 to 1991 were relatively dry in terms of rainfall and uneventful in terms of landslides in Hong Kong. As noted by Malone (2004), the resources for slope safety came under scrutiny during this period and spending on slope upgrading declined. However, the climate of opinion started to reverse dramatically starting from 1992 following the fatal landslide at Baguio Villas. The early 1990s brought a string of wetter than average years, with multiple-fatality landslides and accompanying bad publicity regarding slope safety.

The 1992 Baguio Villas landslide and the large number of landslides triggered by the intense 8 May 1992 rainstorm highlighted two matters of concern: firstly, the Catalogue of Slopes compiled in 1978 was incomplete, with some of the uncatalogued slopes posing a significant hazard to the community; secondly, many private owners were genuinely unaware of their obligations to inspect and maintain slopes under their responsibility.

Taking cognizance of the lessons learnt, the GEO launched a 4-year project to update the Catalogue of Slopes and another project was launched by the Lands Department, at the suggestion of GEO, to delineate the maintenance responsibility of all the registered slopes. The GEO also stepped up its public education and publicity campaigns on the importance of slope maintenance and the responsibility of private owners in relation to undertaking regular maintenance on their own slopes.

The 1994 Kwun Lung Lau Landslide

The 23 July 1994 Kwun Lung Lau landslide involved the sudden collapse of a 100-year old masonry wall that was in good maintenance condition (Figure 7). The full height of the masonry wall, together with the slope above, failed and some 1,000 m$^3$ of debris was released, killing five people and seriously injuring three others on the footpath below. In addition, more than 3,900 residents were temporarily evacuated overnight for fear of collapse of the high-rise buildings they occupied. The masonry wall had a maximum height of 10.6 m and a base width of 0.8 m (which was about one-fifth of that shown in the drawings approved by the Authority in 1965), i.e. a slenderness ratio (height/width) of more than 13. This was exceptionally slender compared with typical masonry walls of a similar construction in Hong Kong, which generally have a slenderness ratio of less than 4. The forensic investigation established that the failure involved buckling and brittle collapse of the thin masonry wall. The collapse was triggered by subsurface infiltration from defective buried drainage systems, which saturated and weakened the soil mass. The state of knowledge at that time was that old masonry walls would fail in a ductile manner following deformation for some time. However, the Kwun Lung Lau masonry wall was in a good condition and yet it failed suddenly with little signs of deformation or distress. The actual failure mechanism of the slender masonry wall was very different to the previous understanding and the assumptions made in conventional stability analysis, and it was therefore investigated in detail. The findings of the technical investigation are summarised by Wong & Ho (1997).

Advanced numerical analyses using the distinct element computer program UDEC were
carried out to assist in the diagnosis of the mechanism and causes of the failure (GEO 1994). The analyses indicated that the masonry structure would fail in a complex mode. The masonry wall was found to bulge initially at about mid-height, accompanied by overturning of the portion of the masonry below this level. These deformation modes combined to lead to tensile failure and consequential sudden reduction of the shear strength of the affected mortar joints. Once tensile or shear failure of mortar joints was initiated, the wall would deform rapidly with instability developing in an uncontrolled manner, resulting in brittle fracture of the masonry wall and failure of the ground behind. Such a complex failure mechanism is not considered in conventional retaining wall analyses, which could be unconservative in the case of thin masonry walls. The local professional practice for the assessment of stability of old masonry walls was duly revised subsequently.

![Figure 7: The 23 July 1994 Kwun Lung Lau landslide](image)

The Kwun Lung Lau landslide also highlighted the critical importance of leakage from underground water-carrying services on slope stability. Finite element seepage analyses were carried out to assess the contribution of the different sources of water in saturating the ground behind the masonry wall. The analyses established that the subsurface seepage led to the wetting up of the loose fill behind the masonry wall. The consequential settlement probably led to distress or rupture of a foulwater sewer running across the upper part of the landslide area, which had a rigid joint that was susceptible to ground deformation. This resulted in substantial saturation of the retained groundmass leading to the collapse.

The vital importance of adverse changes in the environmental setting involving leakage from defective water-carrying services (due to deterioration and poor maintenance) in potentially destabilizing a slope is emphasized by the Kwun Lung Lau landslide. The fatal incident led to the issue of a Code of Practice on Inspection and Maintenance of Water-carrying Services by the Government to upgrade professional practice in the investigation and maintenance of underground water-carrying services (ETWB 2006).

**Observations**

The Kwun Lung Lau landslide disaster aroused grave concern of the public. The rescue efforts continued for days, constantly televised. Slope safety has not gained such prominence in Hong Kong since the 1970s and it became an electoral agenda for the District Board
elections in September of that year. Views were diverse, revealing some serious misunderstandings of landslide risk management.

Public censure ensued when it was made known that the Landslip Preventive Measures (LPM) Programme had only upgraded a small proportion of the slopes in the 1978 Catalogue of Slopes up to that time. The stakeholders were united in calling for substantial injection of resources to accelerate the Government’s long-term LPM Programme, which is focused on upgrading old substandard man-made slopes.

In October 1994, the Legislative Council voted to create a Select Committee to inquire into the circumstances of the landslide and the related issues, which was only the second Select Committee in Hong Kong’s 150-year colonial history. The Select Committee reported its findings in July 1995 (Legislative Council of Hong Kong 1995). The Government also engaged Professor N R Morgenstern to carry out an independent review of the GEO’s technical investigation and report on the adequacy of the Hong Kong Government’s approach to slope safety. The report was completed in November 1994 (Morgenstern 1994).

The Kwun Lung Lau landslide demonstrated that the sudden collapse of a not too sizeable retaining wall adjoining a footpath could result in multiple fatalities. The strong public reaction following the incident highlighted the enhanced public expectation of slope safety in Hong Kong and reflected the dwindling awareness of landslide risk especially during relatively uneventful years.

Following the recommendations of the Slope Safety Review conducted by the then Works Branch (1995), the policy bureau for slope safety, increased resources were provided to the GEO to accelerate the LPM Programme. The Government subsequently pledged that all high-risk old substandard man-made slopes affecting buildings and busy roads would be retrofitted by 2010. In addition, public education and publicity campaigns on slope safety was to be further stepped up to maintain public awareness of landslide risk.

Based on a review of the Kwun Lung Lau landslide, Morgenstern (1994) advocated that a more integrated perspective should be adopted for slope stability studies. In response to this, the GEO launched a systematic landslide investigation programme in 1997. Since then, about 700 landslide inspections have been carried out (out of more than 3,000 landslides examined) and about 200 investigations have been completed.

The 1999 Shek Kip Mei Landslide
On 25 August 1999, a landslide occurred at a cut slope behind two public housing blocks in Shek Kip Mei, an urban area of Hong Kong (Figure 8). Rainfall preceding the failure was intense with a return period of about 30 years. Extensive cracking of the slope and localized detachment of soil mass were observed on the 50 m high soil cut slope which is covered with hard surface. The toe of a significant portion of the cut slope had displaced forward by up to about 1 m locally, but the distressed material did not completely detach from the slope. The distress occurred in two zones, the northern and southern distressed zones respectively. The total volume of distressed material was about 6,000 m$^3$ with depth of the displaced/distressed groundmass up to about 6 m to 8 m. The landslide did not cause any casualties, however three housing blocks and a temple were temporarily evacuated and subsequently permanently evacuated, with more than 1,000 people being affected. In the process of evacuation, an old woman who had lived in the housing estate for a long time committed suicide because of worry on the move to new environment. Immediately following the landslide incident, the
GEO instigated a comprehensive investigation into the failure. The forensic study was carried out by a consultant under the systematic landslide investigation programme and Professor J B Burland (2000) acted as the independent reviewer.

![Figure 8: The 25 August 1999 Shek Kip Mei landslide](image)

The geology of the cut slope comprised predominantly partially weathered medium-grained granite and adverse geological features in the form of persistent sub-horizontal discontinuities were found in the groundmass. Pre-existing tension cracks were also noted at the soil surface below apparently intact chunam cover. These features indicated that slope movements had occurred in the past.

The cut slope was formed along with site formation works for the construction of the four housing blocks in 1954. It was previously assessed by professional geotechnical engineers to be adequately safe on the premise that pore water suction could be sustained in the groundmass. Only very limited slope stabilization works (mainly local replacement of the surface cover and provision of weepholes) were subsequently carried out, and the basic form of the cut slope had not been substantially modified since its formation.

The post-failure investigation (FMSWJV 2000a) established that the landslide was probably caused by the build-up of the transient groundwater conditions following the prolonged intense rainfall that preceded the failure. These resulted principally from water ingress through surface infiltration through defects in the surface cover and subsurface seepage from the regional groundwater flow regime in the bedrock. Water ingress was likely to have led to reduction in suction in the groundmass, development of cleft water pressures in pre-existing tension cracks and elevated water pressures in the discontinuities. The laterally persistent (60 m long) discontinuity with infill of slickensided kaolin and manganese oxide deposits of up to only about 15 mm thick (Figure 9), which was not appreciated in the previous stability assessment, acted as the basal rupture surface in the southern distressed zone. The presence of
extensive, open or infilled tension cracks (up to about 300 mm wide and 1.5 m deep) below the hard surface cover also provided crucial evidence of progressive displacement and slope deterioration (Figure 10).

Figure 9: Laterally-persistent discontinuity infilled with slickensided kaolin and manganese oxide deposits at the 25 August 1999 Shek Kip Mei landslide

Figure 10: Pre-existing tension cracks identified in the 25 August 1999 Shek Kip Mei landslide, (a) completely infilled, (b) partially infilled, and (c) not infilled
Observations

The Shek Kip Mei landslide, together with observations from other notable landslides in Hong Kong, highlighted that slopes are liable to gradual degradation with time (e.g., occurrence of intermittent slope movement during heavy rainfall, formation of local tension cracks or erosion pipes, etc.). Routine maintenance of man-made measures, such as drainage channels and surface cover, will ensure that these will continue to serve their intended functions, as otherwise the slopes will be more vulnerable to local failures and slope degradation may be aggravated, but this cannot prevent progressive degradation of the sloping groundmass.

The Shek Kip Mei landslide further highlighted that unsupported cuts in weathered profiles can be vulnerable to undetected adverse geological and groundwater conditions and can be more prone to slope degradation. This emphasized the need for more robust schemes to enhance slope stability (such as soil nails or retaining walls, together with prescriptive drainage provisions), which are less sensitive to local adverse ground and groundwater conditions.

Given the evidence of progressive slope degradation of the subject slope at Shek Kip Mei since the early 1990s (e.g. repeated cracking of the hard surface cover, minor bulging, etc.), the importance of regular slope inspections and maintenance is further reinforced. This highlighted that slopes that are at a more advanced state of degradation, with signs of distress or with past instabilities, should be targeted for follow-up action under the LPM Programme. The first 3 decades of the LPM Programme have adopted a ‘total retrofit’ approach to deal with the high-risk substandard man-made slopes in the interest of public safety. Capitalising on the lessons learnt from landslide investigations and taking cognizance of the risk profile of the remaining slopes, a new strategy would be adopted by the GEO upon completion of the current phase of the LPM Programme in 2010. This involves targeting approximately the worst 1% of the remaining man-made slopes that are at a more advanced state of degradation for systematic upgrading under a rolling enhancement programme, in a proactive manner. The above new strategy provides a pragmatic and cost effective approach in tackling the remaining large number of man-made slopes that pose a moderate risk to the community.

The Tsing Shan and Sham Tseng San Tsuen Debris Flows

On 11 September 1990, a massive channelised debris flow occurred on the eastern flanks of Tsing Shan (Figure 11). The failure was triggered by relatively light rainstorm with a return period of less than 3 years (Chan et al. 1991). The initial failure was in the form of a relatively small-scale (350 m$^3$) rock topple and rock/soil slide in the source area on the steep upper reaches of the natural hillside. The failed material travelled over an exposed sheeting joint and entered the streamcourse below, developing into a channelised debris flow along the steep drainage line that is infilled with loose bouldery colluvium. Given significant entrainment of loose material along the drainage line, the volume of landslide debris reached approximately 19,000 m$^3$, making it the largest known mobile urban debris flow in Hong Kong to date. The runout distance of the landslide was approximately 1 km, with the debris encroaching on a vacant building platform. Had the housing development proceeded as per the original plan, the consequences could have been very serious.
About ten years later, over 120 natural terrain landslides occurred on the eastern flanks of the Tsing Shan Range during the 14 April 2000 rainstorm when 250 mm of rain fell in 24 hours. Amongst these landslides, the largest one was a channelised debris flow with an estimated volume of about 1,600 m$^3$, which occurred in the early part of the rainstorm with a return period of less than 3 years. The failure was initiated as a debris avalanche and subsequently became a debris flow in a valley adjacent to the 1990 channelised debris flow (Figure 12). Bifurcation of the main debris flow resulted in two relatively smaller debris flows along two separate drainage lines. The source area was pre-disposed to generation of debris flows with the accumulation of loose bouldery deposits on steep slopes above well-incised drainage lines, which are infilled with loose bouldery colluvium (King 2001). Floodwater and some landslide debris reached a construction site at the foot of the hillside, blocking drainage channels and causing washout failures. As a result, some wet debris was deposited on the tracks of the Light Rail and the road beyond, causing socio-economic consequence to the affected community. Evidence of remoulded debris in the failure scar immediately downslope from the source area suggested that the finer part of the displaced material could have rapidly become mobile and moved in the form of slurry flow. The landslide study (King 2001) established that the failure was caused by the build-up of groundwater pressure in the near-surface groundmass during the rainstorm.
Another natural terrain that is also known to be vulnerable to debris flows is the hillside above Sham Tseng San Tsuen. The natural hillside has a history of past instabilities with more than 10 failures occurring during the two severe rainstorms in May and August 1982, the debris of some of which reached the squatter dwellings at the foothill. Further failures on the natural hillside were also triggered by the severe rainstorm in 1999.

On 23 August 1999, four landslides occurred on the natural hillside during an intense rainstorm with a return period of about 50 years. The debris of the largest landslide (600 m$^3$) became channelised upon reaching the streamcourse below the source area (Figure 13), which travelled downslope for a horizontal distance of more than 270 m and demolished a dwelling in Sham Tseng San Tsuen, causing one fatality and 13 injuries. The debris contained abundant boulders of up to 1 m in size. The rocky gully did not contain much loose materials and hence there was little entrainment.

The natural hillside is vulnerable to rain-induced shallow failures. The thin mantle of bouldery colluvium overlying the relatively less permeable saprolite is prone to build-up of perched water pressure during heavy rain. The presence of relict landslides at the 1999 source area probably contributed to disturb and weaken the groundmass and adversely affected the local hydrogeology by promoting direct infiltration of rainwater through open or partially infilled joints. A hillfire that occurred several years before the fatal landslide removed much of the vegetation cover at the source area and its vicinity, which may be a contributory factor.

Figure 12: The 2000 Tsing Shan debris flow
to the failure. There was also evidence of the hillside being subjected to progressive degradation with intermittent slope movements and dilation of relict discontinuities with time. The forensic investigation (FMSWJV 2000b) concluded that the channelized debris flow was rain-induced and the failure was probably caused by elevated groundwater pressure in the near-surface colluvium.

Figure 13: The 23 August 1999 Sham Tseng San Tsuen debris flow

Observations
The Tsing Shan and Sham Tseng San Tsuen landslides highlighted that natural terrain failures, in particular debris flows, can pose a potentially significant risk to the community. The difficulty in predicting the locations of natural hillside failures and the mobility of landslide debris are also emphasized.

Natural terrain landslide risk has been increasing due to more urban developments close to steep hillsides. This is accentuated by the public resistance to reclamations in recent years. Given progressive slope degradation and the vulnerability of many of the marginally stable natural hillsides to shallow rain-induced failures, together with the possibility of extreme weather conditions (e.g. more frequent occurrence of severe rainstorms due to the effects of global warming), much technical development work has been undertaken by the GEO to improve the understanding of the causes and mechanisms of natural terrain landslides. The key areas of work include quantitative risk assessment, rainfall-landslide correlation, hillside susceptibility analyses, debris mobility modeling, customization and application of digital technology, compilation of natural terrain landslide inventories, development of risk-based priority ranking system, regional natural terrain hazard review, design of debris-resisting barriers, etc.

Natural terrain risk mitigation has received increased attention in recent years under the LPM
Programme. Expanded efforts will be made under the post-2010 programme to systematically combat the risk of problematic natural hillside catchments with a history of past failures that are close to buildings and important transport corridors.

The 1990 and the 2000 Tsing Shan debris flows, together with the 1999 Shan Tseng San Tsuen debris flow, highlighted a new area of landslide risk arising from the natural terrain. These large-scale natural hillside failures also highlighted the marginal stability of steep natural hillsides in Hong Kong and the potential hazard of low-frequency, high-magnitude natural terrain landslides, which could result in severe consequences if they were to occur close to densely developed areas.

EVOLUTION OF LANDSLIDE INVESTIGATIONS IN HONG KONG

In Hong Kong, landslide investigations have long played a key role in the advancement of knowledge on slope performance and understanding of the causes and mechanisms of landslides. The arrangement of landslide investigations has, however, evolved with time.

Starting from the 1950s, academics have been involved in studying landslides. For example, for the severe rainstorm in June 1966 which triggered some 500 landslides and resulted in significant loss of life and socio-economic consequence, the slope failures were reviewed by a local academic in the geography department (So 1970). However, the more serious landslides, including the fatal incidents, were not subjected to detailed technical investigations by the Government. Peter Lumb, professor in soil mechanics at the University of Hong Kong, was a key figure in the study of landslides in the 1960s and 1970s (Lumb 1972, 1975), both as part of his research or as an expert engaged by the Government for specific landslide (e.g. the 1970 Fat Kwong Street landslide). In addition, consultants were engaged by the Government to investigate some of more serious landslides (e.g. the 1972 Ching Cheung Road landslide). The findings of the landslide studies during this early period were largely promulgated to the engineering profession through technical papers published in journals and conference proceedings and through functions organised by the local learned societies.

The string of landslide disasters with multiple fatalities in 1972 and 1976 prompted a different approach to the landslide investigation. The Government appointed a Commission of Inquiry to study the two fatal landslides in 1972 and a geotechnical expert from a consultant was also engaged to assist in the technical investigation of the Po Shan Road landslide but not for the Sau Mau Ping failure. Following the recurrence of a similar mobile landslide with multiple fatalities in Sau Mau Ping in 1976, the Government appointed an Independent Review Panel comprising several landslide experts to investigate the fatal collapse. All the above reports were made available to the general public and the process is transparent.

With the establishment of a dedicated geotechnical arm in the Government in 1977, the GEO took a leading role in conducting investigations of serious landslides in-house with a view to advancing the understanding of slope failures. These included studies of selected fatal landslides (e.g. the 1981 boulder fall at King’s Road) for submission to the Coroner’s Court. The findings of the technical investigations were documented in study reports and the observations made were promulgated to the profession. Apart from undertaking landslide studies from time to time, systematic collection of landslide data was also initiated in the early 1980s and annual reports containing factual information on rainfall and landslides have
been published by the GEO since 1984. Reports on overviews of landslides triggered by specific severe rainstorm have also been published by the GEO (e.g. the May and August 1982 rainstorms, the 8 May 1992 rainstorm and the 5 November 1993 rainstorm).

In the early to mid-1990s, the GEO continued to carry out a number of in-house investigations into the fatal landslides, namely the 1992 Baguio Villas landslide and Kennedy Road landslide, the 1993 Cheung Shan Estate landslide, the 1994 Kwun Lung Lau landslide and Castle Peak Road landslide, the 1995 Fei Tsui Road landslide and Shum Wan Road landslide. Apart from the 1994 Kwun Lung Lau landslide and the two fatal landslides in 1995, none of the above forensic investigations involved the engagement of an external independent reviewer.

The systematic landslide investigation programme was launched by the GEO in 1997, initially as a 3-year pilot trial which confirmed its usefulness. It has been integrated with the LPM Programme since 2000. Under this programme, all reported landslides are examined and deserving cases are selected for follow-up studies. The systematic landslide investigation programme is implemented with the assistance of consultants managed by the GEO, which has worked well in meeting the operational needs. Input from overseas landslide experts, in the capacity of independent reviewers, could be tapped through the consultancies.

The main goals of systematic landslide investigations are:

(a) identification of slopes in need of early attention before the situation deteriorates to result in a serious problem;

(b) improvement in knowledge on the causes and mechanisms of landslides so as to formulate new ideas for reducing landslide risk and enhancing the reliability of landslide preventive or slope remedial works;

(c) provision of data for reviewing the performance of the Government’s Slope Safety System and identifying areas for improvement;

(d) provision of evidence in forensic studies of serious landslides that may involve the Coroner’s inquest, legal action or financial dispute.

Apart from study reports on the individual significant landslides, an annual diagnostic report is also prepared, which augments the annual factual report. The annual diagnostic report seeks to make observations on overall trends, consolidate key findings and make recommendations to enhance the slope engineering practice and landslide risk management. In addition, thematic studies (e.g. review of soil-nailed slope failures, review of landslides during construction of slope works, etc.) are carried out from time to time. The quality data and insightful observations made from systematic landslide investigations have provided a valuable source of information to facilitate technical advancements in various areas including landslide debris mobility, quantitative risk assessments, rainfall-landslide correlations, etc.

Experience in Hong Kong has reinforced the advantages of engaging consultants to study landslides and review the performance of the Government’s Slope Safety System by virtue of the impartiality of an independent party. Such independence is particularly of the essence for forensic investigations of fatal landslides from a public accountability point of view. Notwithstanding this, a small number of landslides will continue to be carried out by the
GEO, resources permitting, in order to upkeep the in-house expertise in landslide studies.

The systematic landslide investigation programme will remain an integral part of the long-term Landslip Prevention and Mitigation Programme (LPMitP), which will dovetail with the LPM Programme upon completion of its current phase in 2010, and will be implemented on a rolling basis. With the planned expanded effort in systematic mitigation of natural terrain landslide risk in the years to come, it is anticipated that there would be an increasing reliance on performance monitoring or ‘health’ monitoring of selected sizeable man-made slopes or natural hillside catchments (e.g. large slow-moving landslides), and that some landslide studies may be triggered by the monitoring data to assist in assessing the necessary risk management actions. Another potential new area requiring input from the systematic landslide investigation programme could involve natural terrain risk mitigation measures (e.g. debris-resisting barriers or check dams) following debris or boulder impact. In such circumstances, landslide studies could be initiated to assist in studying the slope failures and reviewing the adequacy of the risk mitigation measures.

CONCLUSIONS
As in many other places, landslides play an important role in the formulation and enhancement of the Slope Safety System and practice in Hong Kong. They have brought about lessons on areas for improvement to our technical, administrative and regulatory provisions. They have helped to remind the stakeholders of the need for continued vigilance in managing landslide risk, and provide opportunities for securing resources for slope safety work. The systematic landslide investigation programme will continue to serve as an integral part of Hong Kong’s initiative to ensure a sustainable slope safety environment as a modern metropolitan and tourist hub.

REFERENCES


**ACKNOWLEDGEMENTS**

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, Government of the Hong Kong Special Administrative Region. The authors would like to acknowledge the assistance provided by Mr. Jonathan Lau in the preparation of this paper.
LANDSLIDE HAZARD MONITORING AND WARNING SYSTEM FOR LI-SHAN AREA

H. L. Wu
Soil and Water Conservation Bureau, COA

M. B. Su
Civil engineering Department, National Chung-Hsing University, Taiwan

Abstract: Due to heavy rains, the sites near the highway 7A (73k+150) and the highway 8 (82k) in the Li-shan township began to subside in mid April 1990. In this study, topography, geology, and groundwater condition of this area were first reviewed. Based on this review, together with field investigations, a general hypothetic model was established to illustrate the Li-shan landslide. Then, a series of limit equilibrium back analyses were performed to understand the failure mechanism of this landslide area. In addition, the performance of the remediation treatment was reviewed. Finally, by combining the automatic monitoring station with internet embedded controller, real time monitoring results can be accessed through internet.

INTRODUCTION

Located at the midway of the east-west cross-island highway (the highway 8), Li-shan is an important small town not only for transportation but also for tourism in central Taiwan (see Figure 1). In mid April 1990, due to heavy rain, the sites near the highway 7A (73k+150) and the highway 8 (82k) began to subside as their foundations are located on the sliding block of the landslide. It is generally suggested that the Li-shan landslide is predominantly caused by heavy rain together with the poor drainage condition.

In order to keep the highway functioning and secure the town for living, the government had executed the first phase emergency treatment followed by the second remediation treatment since July 1990. A dewatering system, including surface ditches, drainage wells and two drainage galleries, was constructed and completed in early 2003. And the effectiveness of the remediation treatment has been strongly revealed as the Li-shan landslide survived the Chi-Chi earthquake (ML=7.3) in 1999.

In view of the characteristics of the sliding area, it is of great interest to understand the complicated failure mechanism, which motivates this study. In this study, topographical, geological, and groundwater conditions of this area were first studied with field investigations. A series of limit equilibrium back analyses were performed to understand the failure mechanism of this landslide for different phases. In addition, the performance of the remediation treatment is also discussed with risk estimation.
Figure 1: A bird eye view of the Li-shan landslide area (looking to the southwest)

Figure 2: The rejuvenation in the Tachiachi river makes the toe steeper than the head

GEOLOGICAL CONDITIONS
In western Taiwan, the westward thrust front due to the compression of the Philippine Sea Plate is obstructed by the rigid basement Peikang High (part of the Eurasian Plate) and results in a series of Quaternary thrust faults trending north-south and dipping towards the east. The 1999 Chi-Chi earthquake is considered as re-activation of one of the Chelungpu thrust fault. However, the Li-shan fault, a major ridge fault of Taiwan island also generated by the above tectonic activity, is located few kilometers west of the Li-shan landslide.

Topographically, Li-shan area is located in the valley of the Tachiachi river. Geologically, Li-shan is not far from the Li-shan fault, and it is suggested the geological conditions is more complicated than expected in this area. And it is evident that the Li-shan is located at colluvial formations originally from the Miocene Li-shan slate formation. The testing results show that the Li-shan slate is about 2.76 ton/m³ of unit weight. And the mechanical properties for different weathering conditions can be summarized in Table 1.
Table 1: Mechanical properties of the geo-materials in Li-shan area

<table>
<thead>
<tr>
<th>Geomaterial type</th>
<th>Unit weight (ton/m³)</th>
<th>Cohesion c (ton/m²)</th>
<th>Friction angle φ (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Colluvium</td>
<td>2.06</td>
<td>0.75</td>
<td>30</td>
</tr>
<tr>
<td>Medium to highly weathered slate</td>
<td>2.69</td>
<td>3.00</td>
<td>28</td>
</tr>
<tr>
<td>Fresh to medium weathered slate</td>
<td>2.70</td>
<td>30.00</td>
<td>33</td>
</tr>
<tr>
<td>Sliding plane</td>
<td>2.69</td>
<td>3.00</td>
<td>28</td>
</tr>
</tbody>
</table>

**MONITORING SYSTEM**

Eight monitoring stations were set up in this area. Each station was equipped with facilities, such as the piezometer for measuring the groundwater level, the inclinometer for monitoring the ground deformation, and the extensometer for detecting the surface movement. Figures of schematic arrangement of automatic monitoring system are given in Figure 3 and Figure 4.

Figure 3: Schematic arrangement of automatic monitoring system applied in Li-shan landslide area
Inclinometers are the devices which are used to detect the location of the slip surfaces in unstable slopes. Two types of inclinometer are used in this study.

**In-place Inclinometer**

The sensor packages are spaced along a standard grooved inclinometer as shown in Figure 5. The sensors are aligned and secured in the casing by spring-loaded wheels, which fit the casing grooves. Readings could be obtained by measuring the change in tilt of the sensor, and then multiply it with the gauge length or spacing between sensors. The results are expressed as the relative displacement of each sensor; and these relative displacements can be summed to determine the total displacement at each sensor. The maximum deflection range is between 15° and 30°, and the precision is 0.01°.
Probe Inclinometer
The probe inclinometer consisted of four components, which includes the guide casing, probe sensor, control cable or wire, and the portable control and readout unit. The probe sensor is lowered on an accurately marked cable, its wheels following the oriented slots of the guide casing. The response to slope changes in the casing could be monitored and recorded on the readout unit manually or automatically.

Figure 6 is the result of recorded data by the automatic monitoring system. It can show the relationship between rainfall, groundwater level and ground surface movement.

![Figure 6: Result of recorded data by the automatic monitoring system](image)

TDR Monitoring System
TDR (Time Domain Reflectometry) technique is applied to monitor sliding within slopes. (Figure 7) When a coaxial cable is embedded in the borehole works like a continuous sensor which can detect fracturing and relative movement at any location along its length. An electromagnetic pulse is launched down the cable and reflection from points of cable deformation can be located precisely. The recorded waveform is shown in Figure 8. TDR monitoring provides a viable tool when location of deformation are not known in advance. This is the major advantage for TDR compared with other monitoring systems. Telemetric monitoring based on TDR theories has been proven to be applicable.

TDR has been applied to monitor the landslide region in Li-shan. The findings indicate that the location of sliding surface detected by using this technique was compared fairly well with the report of the boring-log exploration (Figure 7), in which the sliding surface was found at the interface between the highly weathered slate and the intact rock.

Using TDR coaxial cables to monitor sliding plane is proved to be effective. For landslide happened in highly weathered rock slope in steep mountainside, which is very common in
Taiwan island, there are significant sliding zones or planes existed. Grouted coaxial cables inside borehole can detect sliding much better than traditional inclinometer including in-place type and probe type.

Figure 7: TDR monitoring system
FAILURE MECHANISM

Based on the field investigations together with the topographical and geological information, a general hypothetic model was established to illustrate the Li-shan landslide. This model comprises major factors as below: (1) the sliding planes is basically along the lower boundary of the regolith, about 20m below the surface, (2) there is a major old landslide at the center of the town, (3) the high erosion rate by the streams makes the slopes more dangerous than the others.

The landslide area can be divided to three regions, i.e. the west, northeast, and southeast regions (see Figure 10). Except the southeast region, most of the unstable slopes possess shallow sliding planes at about 9-26 m below ground surface. However, there is an old landslide within the southeast region, of which boundary subdivides the southeast region to two subregions. According to the core logs and the records of drainage gallery construction, the old sliding plane is more than 40-60 m below ground surface. The rest of the southeast region is more or less located at a valley of a small branch of the Tachiachi river. Due to the tectonic activities, there is rejuvenation in the Tachiachi river. Thus the erosion rate of this branch is quite high, which generates higher hazard potential of this subregion (see Figure 2).

Based on the study of topography, the profiles AA’, BB’ and CC’ (see Figure 11) were adopted and analyzed by the limit equilibrium analysis model PC-STABL. The slopes are fairly stable for dry condition as the safety factor is 1.21-1.35; but they become critical with high groundwater level as the safety factor drops to 0.99-1.15. This finding might reveal that there are more than one activities in this area, as the precipitation is quite high in this area, about 2340 mm annually. Therefore, it is essential to have remediation treatment in this area (see Table 2). Besides, during the Chi-Chi earthquake, the horizontal acceleration was estimated to be 0.15-0.20g. With this impact, this area survived except minor damages near the profile BB’. It somehow reveals the effectiveness of the remediation.
As the landslide is closely related with the rainfall and groundwater, groundwater control is essential for slope stabilization in this area. A drainage system, composed of surface and subsurface drainage components, was designed to be a remediation treatment. For the surface drainage system, existing ditches were integrated as a system to divert the undesirable surface flows into non-problematic areas, as well as to prevent excessive water infiltration near tension cracks. In order to more efficiently control the groundwater level, a subsurface drainage system is also applied, which is consisted of three major components, i.e. horizontal drainage pipe, drainage well, and drainage gallery. There are (1) 15 horizontal drainage sites, 7-9 pipes (30-60m in length) in each site, (2) 13 drainage wells, located mainly in the heads of slopes, and (3) 2 drainage galleries, excavated below the sliding planes. The surface and subsurface drainage systems are illustrated in Figure 12.

**Table 2: Safety factors for the residual slopes in Li-shan area**

<table>
<thead>
<tr>
<th>Profile analyzed</th>
<th>A-A’</th>
<th>B-B’</th>
<th>C-C’</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry (no groundwater)</td>
<td>1.23</td>
<td>1.21</td>
<td>1.35</td>
</tr>
<tr>
<td>Wet (high GWL)</td>
<td>1.11</td>
<td>0.99</td>
<td>1.15</td>
</tr>
<tr>
<td>Wet (with remediation)</td>
<td>1.23</td>
<td>1.18</td>
<td>1.22</td>
</tr>
</tbody>
</table>

**REMEDICATION**

As the landslide is closely related with the rainfall and groundwater, groundwater control is essential for slope stabilization in this area. A drainage system, composed of surface and subsurface drainage components, was designed to be a remediation treatment. For the surface drainage system, existing ditches were integrated as a system to divert the undesirable surface flows into non-problematic areas, as well as to prevent excessive water infiltration near tension cracks. In order to more efficiently control the groundwater level, a subsurface drainage system is also applied, which is consisted of three major components, i.e. horizontal drainage pipe, drainage well, and drainage gallery. There are (1) 15 horizontal drainage sites, 7-9 pipes (30-60m in length) in each site, (2) 13 drainage wells, located mainly in the heads of slopes, and (3) 2 drainage galleries, excavated below the sliding planes. The surface and subsurface drainage systems are illustrated in Figure 12.
From the preliminary results of groundwater level monitoring, the groundwater level has been successfully reduced about 10-20m after the drainage gallery No.1 in operation. By this improvement, the stability of slopes is reasonably improved as expected with safety factors of around 1.18-1.23.

INTERNET EMBEDDED PREWARNING SYSTEM
By combining the automatic monitoring stations with internet embedded controller (Figure 13), the system is reconfigured into an internet server based system. Real time monitoring results can be accessed through internet (Figure 14). A GIS database server collect data from field station to calculate factor of safety for slope against sliding. Using the criterion discussed above, the judgement can be made easily. And, decisions for response in regard to local residents' safety can be made by computer automatically.
Figure 13: Automatic monitoring system

Figure 14: Internet page (Real time monitoring and prewarning)

CONCLUSIONS
Stability analyses were employed to study the behavior of the slope and the failure mechanism of the Li-shan landslide. The slopes are quite stable for dry condition, but become critical for fully saturated condition. The remediation treatment is essential, as the precipitation in this area is quite high in this area. According to the renovation proposal
"Investigation and Renovation Planning for Landslides in Li-Shan Area" by Industrial Technology Research Institute, the groundwater level could be lowered by 8.3 m after the remediation work is accomplished. In fact, the groundwater level was dropped by an average of 12 m, measured by the 2nd Engineering Office of Soil and Water Conservation Bureau on the monitoring stations, which was better than the previous estimation. The risk reduces to 1.0 % when counted on the contribution of the remediation treatments.

REFERENCES
REPORT ON FORENSIC LANDSLIDE INVESTIGATIONS

Luciano Picarelli
Dipartimento di Ingegneria Civile, Seconda Università di Napoli

The Session 2 on “Landslide Investigations” included seven papers concerning both single cases, which were used to discuss causes and mechanisms of slope movements (first half of the session), and catastrophic landslides in exposed areas, which gave the opportunity to debate about land planning and risk mitigation (second half of the session).

The four papers selected for the first part of the session (single cases) concerned:

- the Aznalcóllar landslide in Spain (presented by A. Gens);
- the Storrega landslide in the Northern Atlantic Sea (presented by F. Nadim);
- the Li-shan landslide in Taiwan (presented by M.B. Su);
- three cases of failure of cut slopes in Korea (presented by S.G. Lee).

The discussion included, first, questions from the room to the speakers, then a debate about the following couple of topics proposed by the Chairman, L. Picarelli:

1. What kind of investigations should be carried out and which soil parameters should be accounted to predict catastrophic landslides?

2. When can a slow landslide turn into a fast one?

The discussion was conducted with the support of two panelists, A. Gens and O. Hungr, previously selected by the Chairman.

A significant part of the time allocated for questions from the room concerned the Aznalcóllar landslide, a nice and well documented example of progressive failure of a tailings dam, involving part of a pre-sheared bedding surface present in the subsoil. Progressive failure is a well known problem, which has been highlighted by some fundamental papers written by Bjerrum, by Burland, Longwoorth and Moore, and by Skempton and Potts, and the case described by Gens represents one of the very few prominent additional contributions to the topic. Unfortunately, despite progressive failure being probably the norm for the majority of slope failures, appropriate slope stability analysis is still extremely difficult because of a number of uncertainties and constraints in the numerical procedures.

The importance of pre-sheared bedding planes, of the construction procedure and of the excess pore pressures induced by the dam, in the triggering of the landslide, was outlined by the speaker and by several contributions (D. Chan, J. Hutchinson, A. Leventhal, L. Picarelli, O. Hungr and K. Ho). The need for good geological investigations prior to design and construction and the role of the residual shear strength in progressive failure were also stressed.

The historic Storrega landslide which occurred in past times in the Northern Atlantic Sea, not far from the Norwegian coasts, represents an important case of submarine landslide. A retrogressive movement was the most likely mechanism of the landslide, which was probably
provoked by rapid sediment accumulation from glacial deposits and the consequent build up of excess pore water pressures. Probably a strong earthquake contributed to the triggering of the landslide. The Storrega landslide is a key case for analysis of seabed stability, especially in those parts of the world that are or can be exploited for gas or oil production.

E. Leroi and O. Hungr contributed to the discussion, posing questions about the role of the chemical composition of the sediments in the failure, and about the implications in the exploitation of the gas field of Ormen Lange, which is very close to the scars of the Storegga landslide.

The Li-shan landslide involves weathered slate, which can be exacerbated by adverse meteorological conditions. For this reason, the area is continuously monitored and a pre-warning system has been installed. The paper presented by M.B. Su concentrated essentially on the features of the monitoring system which includes some advanced procedures, such as the use of TDR for investigation of slope displacements at depth.

The last presentation concerned the failure of three cut slopes in rock. The speaker highlighted the need for an adequate ground investigation and proper design standards to avoid failures of high cuts, which are still rather frequent in Korea. This is an important point because an inappropriate approach in the design and construction of high cuts is unfortunately rather widespread round the world.

The general discussion on the two topics proposed by the Chairman involved several discussers (R. Poisel, O. Hungr, F. Nadim, J. Hutchinson, A. Gens and L. Picarelli). A general agreement concerns the fact that the transition from a slow to a fast landslide and the consequential impact on exposed elements depends on the mechanism of slope deformation, which can change during movement and often leads a drained deformation process to an undrained process. In this respect, the role of geological structures and of soil properties is important. O. Hungr, for example, suggested that the presence of weak and strong elements (such as weak joints within rock masses) in a slope can play an adverse role. L. Picarelli, in turn, mentioned the quite frequent case of slow moving mudslide bodies that are subjected to the accumulation of debris discharged by smaller landslides triggered along its main scarp and along lateral boundaries (alimentation), causing the build up of positive excess pore pressures (through undrained loading) and the acceleration of slope movement. As outlined by some discussers (J. Hutchinson and O. Hungr), the prediction of the magnitude of a landslide (e.g. size and velocity) is a rather complex task, but investigations focusing on the evaluation of what could happen if a given mechanism was triggered can be extremely useful. In other words, the designer or the consultant should have in advance a clear idea about the possible challenges inherent in a given geomorphological situation, directing the investigations to examine if the scenarios he suspects as being possible in the future life of the slope can really happen.

The papers selected for the second part of the discussion (catastrophic landslides in exposed areas) concerned:

- landslides in Malaysia (presented by A. Othman);
- the Sarno debris flows in Italy (presented by P. Versace);
- milestone landslides in Hong Kong (presented by H.N. Wong).

The panellists were F. Nadim, P. Versace and H.N. Wong. The topics proposed for the debate were the following:
1. What is the way to integrate investigation in landslide prone areas, given the framework of experience?

2. Can early warning be confidently used for mitigation of the risk of rapid landslides in urban areas?

The first presentation was delivered by A. Othman who described four recent catastrophic landslides in Malaysia involving different materials and displaying very different features. The causes and mechanisms of such landslides were discussed in detail.

The catastrophic Sarno experience, in Italy, is quite recent and well known around the world: several tens of large debris flows were triggered all around the Pizzo d’Alvano mountain in a time span of a few hours (May 1998), hitting four villages and killing 159 people. The mobilised soils were pyroclastic materials (alternating layers of ash and pumice) deposited some thousands of years ago as a consequence of volcanic activity. These rainfall-induced landslide events are not new, as old chronicles testify, but in recent years their impact has grown enormously because of the increase in the number and extent of developed areas. Fortunately, in a few years the research provided new and significant insights and results, which clarified the triggering mechanisms of debris flows, and supported the implementation of efficient active and passive measures for risk mitigation. The measures adopted in the Sarno area were the main objective of the presentation by P. Versace.

Several questions were posed to the speaker concerning the return period of such events (M. Sheridan, E. Leroi) and the assessment of the efficiency of the adopted measures (A. Leventhal). The assessment of the frequency of landslides in a given area is a complex problem, which is much more complex than the equivalent problem regarding other natural phenomena such as floods. According to historical data on landslides, the return period of debris flows in the Sarno area is about a hundred years. However, this does not concern single slopes or catchments, but wider areas where the same materials outcrop. Fortunately, the efficiency of the mitigation measures adopted in Sarno has not been tested by new landslides. However, extensive slope monitoring provides continuous information about the effects of rainfall on slope behaviour.

The fight of the Hong Kong Authorities against landslides is well known all around the world. Hong Kong is a leading country in this field concerning the results of the research and, mostly, the efficiency of public organization in risk mitigation. Based on a general review of the key milestone landslides in Hong Kong and of the subsequent measures taken by the Authorities, the third paper discussed the lessons learned through experience and the outcome of the establishment of the present system of slope stability analysis and control as well as in the impact on the regulatory regime and professional practice. Wong’s presentation raised different questions posed by A. Leventhal, Su-Gon Lee and E. Leroi, regarding the role of GEO, the criteria adopted for risk mapping and slope stability analysis, and the impact of GEO’s activities on the general public. H.N. Wong stressed that still today a deterministic approach is used in the analysis of man-made slopes (the factor of safety must be higher than the stipulated value), but QRA is also adopted for natural hillsides. He reminded that one of the activities of GEO is also to guide the law-makers in the implementation of some of the technical recommendations which result from its activities.

The last part of the session focused on the questions posed by the Chairman with particular reference to landslide alert. Contributions to the discussion came from Hak-Moon Kim.
H.N. Wong, F. Nadim and P. Versace. H.N. Wong highlighted that the strategy adopted in such a densely populated areas as Hong Kong is the stabilization of specific landslide prone slopes where known (e.g. with significant signs of distress) rather than reliance on early warning systems. P. Versace stressed the importance of early warning systems which also have the goal of alerting the civil protection, but noted that these must be used with extreme caution because of the risk of missing the alarms. Certainly, population density is an important factor in the selection of the best criteria for landslide prediction and alerting, because they have to be related to the consequent impact, in case of missing alarms. Finally, the role of the technological improvement in the use of early warning systems, together with the lack of focus on the human element and political considerations in the design and implementation of an early warning system (e.g. changes in human responses following false alarms), was outlined by F. Nadim.

The session on landslide investigations covered a fairly wide range of scenarios concerning both geomaterials and landslide types as well as the approaches adopted in investigations, selection of remedial measures and landslide prevention. Very interesting cases were presented, whose lessons are of high value for researchers and practitioners. Traditional criteria for slope investigation are still used worldwide, but innovative procedures of monitoring and criteria for data interpretation are taking place in many parts of the world. It is clear that the problem of protection of populated areas is growing because of the increasing exploitation of sloping grounds and of the increase of exposure due to the growth of population and economic activity. In addition, climatic changes are posing new challenges and problems. Fortunately the increase in our technical knowledge is significant and still more important is the sharing of experience facilitated by globalization.
SESSION ON FORENSIC LANDSLIDE INVESTIGATIONS:
RECORD OF DISCUSSION

Julian Kwan and H. W. Sun
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

FIRST PART OF THE SESSION

Topics for Discussion

1. What investigations should be carried out and what soil parameters should be determined to predict catastrophic landslides?

2. When a slow landslide can turn into a fast one?

Record of Discussion

Dave Chan:
Significant increase in the pore water pressure was measured at the Aznalcóllar dam, was there any design review before the failure?

Antonio Gens:
The pore water pressure was measured after the dam failure. So, no design review was carried out before the failure.

Jean Hutchinson:
Was there any correlation between the dam height and the pore water pressure?

Antonio Gens:
No.

Andrew Leventhal:
Was there any impact to the design practice of dam after the progressive failure of Aznalcóllar dam?

Antonio Gens:
Improvement to the design practice is needed. Progressive failure mechanism should be considered in dam design. In particular, attention should be given to materials with low residual strength.

Luciano Picarelli:
The failure mechanism of Aznalcóllar dam was complicated since it involved progressive failure in bedding plane. Did John Burland say something about $\phi_{\text{crit}}$ and $\phi_{\text{res}}$?

Antonio Gens:
This is probably a coincidence that values of $\phi_{\text{crit}}$ and $\phi_{\text{res}}$ are similar. It is fundamental and John Burland would not say anything about the point that $\phi_{\text{crit}}$ is similar to $\phi_{\text{res}}$. It probably
comes from Skempton as he noted that values of $\phi_{\text{crit}}$ were apparently close to $\phi_{\text{res}}$.

Oldrich Hungr:
Was there a geological reason for the pre-shearing in the clay layer?

Antonio Gens:
It may be related to tectonic movements.

Oldrich Hungr / Antonio Gens:
There should be a testing programme to identify this kind of adverse features in ground investigation.

Ken Ho:
What was the trigger of the failure?

Antonio Gens:
The exact trigger is not known. The level of the tailing was being raised. There was no rain and obvious signs of distress on the day before the failure.

Eric Leroi:
About the submarine landslide at Storegga, could the trigger be related to the chemical composition of the clay?

Farrokh Nadim:
There could be many possible reasons for the landslide. However, since the landslide happened thousands years ago, the actual situation had been changed and we are not able to trace the exact cause of the landslide.

Eric Leroi:
Is there any impact on the oil production because of the ancient landslide?

Farrokh Nadim:
No.

Oldrich Hungr:
This landslide is huge and stands out in terms of size in the landslide inventory.

Farrokh Nadim:
Agreed. This is an ancient submarine event. The smaller ones that occurred in the same period would not last this long.

Antonio Gens:
Regarding the progressive failure of Aznalcóllar dam, since soil samples collected at the landslide site were disturbed, accurate measurements of the soil strength was difficult.

Oldrich Hungr:
Geological input for assessment of the geological model, including geological structures, is also very important. Investigation should not focus only on mechanical behaviour of samples.
Luciano Picarelli:
Investigation/consideration of brittleness of the material in both drained and undrained conditions are also the important factors, which could give rise to very fast-moving landslides and more severe consequences.

Oldrich Hungr:
Robin Fell, in his Glossop lecture, gave the possible causes of landslides. These include abrupt changes in unbalanced force due to rapid reduction of shear strength. This could be due to the presence of weak and strong elements in the ground (e.g. weak joints in rocks). Undrained failure that leads to loss of shear strength due to liquefaction can trigger a change of mechanism and generate a fast-moving landslide. Similarly, the same can happen in the case of loss of suction/cohesion.

Luciano Picarelli:
The change in unbalanced force condition could be a result of increase in pore water pressure.

Going back to the second question, when can a slow landslide turn into a fast-moving one (e.g. a ductile landslide turning into a brittle one)?

Oldrich Hungr:
It is uncommon to have a slow landslide turning into a fast-moving one, as the fast-moving ones usually have the capability to become fast. There could actually be pre-failure slow movements before the on-set of a fast-moving landslide. The only exception, however, is where certain mechanisms exist that break up the structure of the material over a certain period of time, a slow-moving landslide could turn into a fast-moving one. For example, a 1996 landslide in California which was a slow earthflow turned into a flowslide in 2005.

Luciano Picarelli:
The driving forces can change over time and turn a slow-moving landslide into a fast-moving one.

Rainer Poisel:
Where the landslide mechanism changes, a slow-moving landslide can turn into a fast-moving one.

Mike Winter:
Robin Fell’s Glossop lecture is published in the Quarterly Journal of the Engineering Geology and Hydrogeology and is available on the website of the Geological Society.

Farrokh Nadim:
Hungr made a good point about the conditions for a catastrophic landslide to occur. What investigation do we need to find these conditions?

Oldrich Hungr:
The most important one is geological input in the investigation. We need to have a clear picture of the geology before formulating other models.

Jean Hutchinson:
My question leads on question 1. Until a catastrophic landslide occurs, we don’t know it is catastrophic. Pre-failure investigations for those cases are often insufficient. What we need to do will depend on what do we expect would happen. The investigation should be
done in stages to address these questions.

**Antonio Gens:**
It is important to recognise a problem that may exist and to establish it. Understanding of the geological setting is important in directing a ground investigation, as suggested by Oldrich. On the issue of a slow landslide turning into a fast-moving one, it is important to recognise the drained/undrained conditions given that fast landslides would involve undrained behaviour. Effective stress analyses would miss this important point in the case of fast-moving landslides.

**Jean Hutchinson:**
Could I have Oldrich answer my question, especially on knowledge of geological model?

**Oldrich Hungr:**
It is important to have adequate geological training for engineers. Many geotechnical failures can be traced to over simplification in our soil mechanics textbooks. Conservative assumptions without an adequate understanding on geology will not be adequate.

**Antonio Gens:**
I would like to add a further example on the drawbacks in over simplifications in our soil mechanics textbooks. The use of ‘groundwater table’ is horrible. It is rare to have hydrostatic condition in our groundwater. It is important to recognise pore pressure on potential failure surfaces. This cannot be achieved from water table assumptions given in our soil mechanics textbooks.

**Oldrich Hungr:**
We need to teach our students about the difference between a phreatic surface and piezometric surface.

**Manuel Pastor:**
We heard experience of many participants here. Can we draw a framework of their experience on previous landslide disasters? And how can we integrate the future landslide investigations within the framework?

**SECOND PART OF THE SESSION**

**Topics for Discussion**

1. According to the speakers, experience of previous landslide disasters can practically be used as a tool for prevention of future disasters: what is the way to integrate future investigations in the framework of experience?

2. Early warning can be confidently used for mitigation of the risk of rapid landslides in urban areas?

**Record of Discussion**

**Andrew Leventhal:**
What is Geotechnical Engineering Office (GEO)’s role during building planning stage?
**HN Wong:**
We exercise geotechnical control in all development projects. At the planning stage, GEO would object to the proposal if the site is affected by major landslide hazards and is found to be geotechnically not feasible. Detailed design and construction will also be checked and audited by the GEO. Under the law, all geotechnical construction works would need to have GEO’s agreement before commencement. There is a range of control mechanisms for different situations. In areas where the potential landslide hazards are high (e.g. Scheduled Areas), we have stringent control on bulk excavations and requirements for building structures to withstand impact of landslide debris.

**Andrew Leventhal:**
In the geotechnical design practice of Hong Kong, do designer apply the factor of safety approach or the probabilistic approach?

**HN Wong:**
We adopt the factor of safety approach for design of man-made slopes. The required minimum factor of safety is 1.4. However, we assess, for example, natural terrain hazards using the quantitative risk assessment (QRA) approach. We also use the design event approach for our natural terrain landslide mitigation measures.

**Su-Gon Lee:**
How do you do 10% detailed face mapping in your landslide studies? How can you ensure the quality of the mapping.

**HN Wong:**
On average, 10% of the landslides reported to the GEO were selected for detailed studies. We study important failures.

**Su-Gon Lee:**
What happen if you found out an error was made by the slope designers in your investigation? It is difficult to judge on the quality of face mapping carried out by designers.

**HN Wong:**
In theory, our investigation can lead to legal proceedings against the parties in default. In practice, we aim to identify areas for improvement in our system and practice. Legal action is very rare.

**Eric Leroi:**
How did you explain the F-N curve concept to public?

**HN Wong:**
We have not explained this to the general public. We managed to explain the F-N curve concept to our law-makers. They understand this Okay. This concept has been used previously for potentially hazardous installations (PHI) sites and they have some background knowledge about it. We have not tried to explain the F-N concept to the general public. Our risk acceptance guideline is still an interim one.

**Dave Chan:**
With geotechnical control exercised centrally, what GEO does may stifle innovation.
HN Wong:
This is the worry of some, especially when we have a technically strong and powerful organisation. In practice, we are partnering with the practitioners to tackle landslide problems in Hong Kong.

Michael Sheridan:
What is the return period of the Sarno debris flow? What inventory do you have?

Pasquale Versace:
The return period is estimated to be 100 years. Up to 100 years, we have no documented data.

Andrew Leventhal:
Have your system of mitigation works been tested, i.e. subjected to a debris flow event?

Pasquale Versace:
We just have measurement of rainfall and suction in our slopes. We have model tests on infiltration, etc.

Eric Leroi:
I am not confident with your estimated return period. Do you mean a return period for a region or a site? Do you expect similar events to happen again?

Pasquale Versace:
In many of the smaller areas, smaller landslides are reactivation of landslide debris. It is possible that similar events will happen. There are many small size debris flows happening in the area. In a given rainstorm, some catchments had many debris flows while other similar catchments in the region had very little failures.

Eric Leroi:
It is difficult to predict the occurrence of debris flow, in particular, in the areas where debris flows do not occur very often. We could invest a lot in areas affected by previous major landslides but the potential for future landslides could be relatively low. It is difficult to explain this to the public.

Pasquale Versace:
It can be a political consideration. For example, in the reconstruction of the town of Sarno, the population wants to stay in their home town and we can only provide mitigation works to protect the population.

Eric Leroi:
How much does it cost?

Pasquale Versace:
About 300M euro.

Hak-Moon Kim:
Regarding the topic of using landslide warning in urban areas, we need to ask the question: how can we predict the time of failure of the slopes? We need to enhance our understanding in this aspect, for example, through monitoring moving slopes with instruments.
HN Wong:
We are not good at predicting landslide failure time, even for slopes with signs of distress. In a densely populated area like Hong Kong, early warning system based on prediction of the failure time for evacuation is not a preferred approach. We prefer to fix slopes instead of monitoring.

Hak-Moon Kim:
There are development of using creep concept and instrumentation for monitoring.

HN Wong:
Our densely developed setting does not allow this and we are not good at using these to safeguard public safety.

Farrokh Nadim:
The application of early warning system as mitigation measures has focused on technological development. We have not done much on the social science aspect. HN has shown us this issue for a densely populated city like Hong Kong. It is beyond geotechnology.

HN Wong:
If there is a landslide behind a high-rise building, even if the early warning system works in giving fore-warning for the evacuation of people and saving lives, the landslide will still damage the building and there would be a major public outcry. The buildings are also very expensive compared with the cost of fixing the slopes.

Pasquale Versace:
It depends on the situation, e.g. population density. The early warning system is not only for warning the public but to alert our response system for civil protection. In situation of less densely populated areas, we could rely on the system. There is a risk in this system. Trust from the public is also important. Sometimes, it is better not to have the system if it gives many false alarms.

Farrokh Nadim:
Hong Kong is a model for us. They are able to communicate to the public, to identify the most risky slopes and to focus their efforts on these.

Pasquale Versace:
We have to translate this experience into different situations, e.g. covering a bigger country. Different approaches are needed for different situations.

HN Wong:
I agree. One point I would like to make is that we need the best person to do the landslide investigation.
Session 3  Innovative and Digital Technology
PREDICTING SLOPE FAILURE USING REAL-TIME MONITORING TECHNOLOGY AND THE TRS SENSOR

K. T. Chang
Department of Civil Engineering
Kumoh National University of Technology, Korea

H. S. Han
Department of Civil Engineering
Kumoh National University of Technology, Korea

J. F. Wang
Institute of Mechanics, Chinese Academy of Sciences, Beijing, China

Albert Ho
Ove Arup & Partners Hong Kong Ltd

Devon Mothersille
Geoserve Global Ltd, London, UK

Abstract: The paper summarises research work, carried out to develop a qualified data analysis system for predicting the behaviour and failure of slopes. The data used in the study were acquired from TRS (translation, rotation and settlement) sensors and were transmitted to a control centre using real-time monitoring (RTM) technology, utilizing the CDMA communication system. The RTM system also incorporates the innovative intelligent camera technology, used to observe incipient failure.

The use of appropriate mathematical modelling has been useful in creating a predictive capacity for slope failures. Through observation and analysis of a real-time measured time series, a mathematical model was selected which predicted landslide behaviour with commendable accuracy. Two theoretical models are suggested: the polynomial function and the growth model. These models are judged to be most suitable for the description and analysis of measured deformation from an active landslide. This paper describes the application of these models to field data extracted from a slope in Nerupjae, South Korea. Analysis of the results confirmed good correlation between measured field data and predictive models.

INTRODUCTION

Throughout the years, geotechnical engineers have routinely collected considerable amounts of data derived from the monitoring of slopes prone to failure. Yet despite this vast accumulation of information, the effective use of geotechnical data to facilitate the prediction of behaviour has not been adequately addressed. The frequent occurrence of catastrophic landslides in recent years in Korea has reiterated the urgent need for geotechnical engineers to forecast, more accurately, the slope failure. A review of current practice shows that existing predictive methods for landslides have seldom been based on basic physical principles or concepts. However, observations have confirmed repeatedly that the classical growth models simulating the phenomenon of growth could be applicable in providing a predictive capacity.
For example, the deformation of a rock avalanche grows exponentially or with a power function, over a time period. According to a model devised by Verhulst (1838), the deformation of a landslide with the foot of the slope facing a river may follow the logistic distribution. The present paper shows the methodology involved in utilizing two kinds of theoretical models for the description and analysis of monitoring data and how the resulting information can be interpreted to facilitate prediction.

**PREREQUISITES FOR PREDICTION MODELLING**

The selection of an appropriate mathematical model is vital for the accurate prediction of landslides. It would be erroneous to assume that a particular model is suitable for the prediction behaviour solely on the basis of best fit to data acquired in the field. It would be equally misleading to assume suitability if the residual sum of squares is a minimum between the data and the model. It is important to understand fitting and prediction are entirely different concepts. Fitting provides an indication of an ability to model past and present behaviour and a good fit suggests a satisfactory interpolated estimation. It could be erroneous to extrapolate such an interpolation to the prediction of future behaviour.

A more reliable way to achieve greater accuracy in prediction, is to acquire a better fundamental understanding of the movement mechanisms involved and the nature of the landslide itself. As an example these can take the form of a linearity tendency, a periodical fluctuation, a seasonal transformation, a growing tendency or some form of function described by differential equations. Through the effective use of observation and the analysis of a real-time measured time series, it is possible to select a mathematical model that reasonably predicts the behaviour of a landslide. By fitting the suggested model to the raw data and adopting other relevant parameters in the model, the prediction of failure time can be achieved. (Saito et al. 1961; Voight 1988).

**TWO PROPOSED MODELS**

Experience has shown that the deformation for a landslide with the characteristics of an avalanche is generally represented as straightforward accelerated failure, with little or no inherent or natural constraint. The deformation will appear to follow exponential, power or polynomial growth. On the other hand, the deformation of a landslide into a riverbed, or subjected to unavoidable inherent or natural constraints, approximates to an S-shaped curve. Both types of failure have inflection points and maximum curvature points in their respective deformation curves as shown in Figure 1.

For predicting a landslide based on deformation observed in the early stage, the determination of an appropriate model and its best-fit parameters is still a frequently used method. The key is to select a best-fitting model, considering aspects of engineering geology and minimizing the squared errors between model and the data.

Two models are proposed: 3rd-order polynomial model and the growth or S-shaped model:
For the straightforward accelerated failure case (Polynomial Model), we assume the time function of the deformation, $N$, as:

$$N(t) = a_3 t^3 + a_2 t^2 + a_1 t + a_0$$  \[1\]

The coefficients ($a_3$, $a_2$, $a_1$, $a_0$) could be determined by curve fitting technique using a spreadsheet. The next step in the analysis is to determine the asymptote and maximum points in this curve. In this case, the asymptote indicates the failure of the slope. Since the asymptote is the infinity of the deformation curve of slope, the maximum deformation of slope results in failure. As the profile of the curve approaches the point of rapid gradient change, then it is assumed that landslide will be imminent. To ascertain the point of curve slope change, Eq. [2] is rearranged,

$$\frac{dN}{dt} = 3a_3 t^2 + 2a_2 t + a_1$$  \[2\]

For the failure with inherent case (Growth Model), we also assume the time function of the deformation, $N$, as

$$N(t) = a_7 t^3 + a_6 t^2 + a_5 t + a_4$$  \[3\]

In this case, the value of $a_7$ has a negative value as compared to $a_3$ of the polynomial model. This model also has an asymptote, therefore, the next step in the analysis is to determine the asymptote and maximum points in this curve. As in the previous case, the asymptote indicates failure of the slope. At the point of curve gradient change, landslide warnings should be alerted.
THE MONITORING SYSTEM

Figure 2 shows the specially developed measurement transducer called the TRS sensor. It is designed to measure specific parameters, namely displacement (translation), rotation and settlement (hence TRS). By adding the specific measuring sensor for rotation, the TRS sensor could form the 3D vector which facilitates the understanding of slope movement and tendency. The TRS sensor applied during the Nerupjae case study is shown in Figure 3.

Figure 2: TRS sensors

Figure 3: TRS sensor applied in Nerupjae

A total of 16 nos. of TRS sensors and one rain gauge were installed to analyze the slope behaviour in Nerupjae, Jaechon, which is a cut slope adjacent to a national road. The deformation shape versus time of Nerupjae has followed the typical 3rd-order polynomial equation. Figure 4 shows the computer display together with the associated deformation graph. The data was analyzed, and the results from sensor No. 12 exhibit the typical failure pattern. It is a 3rd-order polynomial function and the deformation equation describing the trend line with respect to time is given as:

\[ y = 10^{-8} x^3 - 3 \times 10^{-5} x + 0.0316 x - 0.3472 \text{ and } R^2 = 0.929 \]  \[4\]

As can be seen in Figure 5, the trend line of the slope deformation is very close to the asymptote, this suggests that the failure of the slope is imminent.
THE INTELLIGENT CAMERA SYSTEM
In addition to the features described, the monitoring system allows the user to establish threshold values for movements such that, if levels are exceeded, warning signals will be issued (see Figure 6).

Visualisation of slope movements using a network camera system is a recent development. The system is designed so that activation of the camera, with sophisticated image processing facilities, occurs when preset threshold levels are exceeded. Additional features include the automatic control of traffic control signals which reduce the risk of accidents and allow the rapid enforcement of countermeasures (see Figures 7 and 8).
As of Nov. 2007, 129 instrumented sites are being monitored in South Korea and these form a substantial database of information which continues to assist with the development of the real-time monitoring system (refer to the Figure 9).
CONCLUSIONS

When compared with other classical mathematical models, the presented model is a better representation of slope behaviour, especially when represented by the polynomial function. Similarly, it was found that \(\frac{dD}{dt}\) and the asymptote in the curve perform key roles in predicting the landslide. Again, compared with other existing models, the suggested models are more generalized and simple since they are 3rd-order polynomial functions.

However, there are still influential factors not incorporated in the presented models, these include geometric shape, amount of precipitation, etc. It can also be said that on occasions the presented models show some instability. The model only considers the data derived from the tertiary creeping stage and not data from the first and second stages. In addition, the physical meaning of the \(a\) coefficient is not yet clear. This suggests that there are still problems and questions to be investigated in the future.

For the slope at Nerupjae, the equation of the deformation trend line with respect to time is represented by the 3rd-order polynomials \(y = ax^3 + bx^2 + cx + d\).

As demonstrated in Table 1, the trend line is judged to be satisfactory due to the high \(R^2\) value.
Table 1: Summary of Nerupjae

<table>
<thead>
<tr>
<th>Field</th>
<th>Failure model</th>
<th>Trend line</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nerupjae</td>
<td>Polynomial</td>
<td>$10^8 x^3 - 3 \times 10^5 x + 0.0316 x - 0.3472$</td>
<td>0.929</td>
</tr>
</tbody>
</table>

The coefficient, $a$, within the Nerupjae equation is a positive value, therefore, the asymptote of Nerupjae curve is located on the $x$-axis (time-axis), also since the trend line of the field data was tending towards the asymptote, it was judged that failure of the slope was imminent.

REFERENCES
FROM PERMANENT SCATTERERS TO WEBGIS: INNOVATIVE TECHNIQUES FOR LANDSLIDE MONITORING AND EARLY WARNING

Giacomo Falorni, Paolo Canuti, Paolo Farina, Lorenzo Leoni and Nicola Casagli
Department of Earth Sciences, University of Firenze

Abstract: The techniques and methods for monitoring relatively slow-moving landslides differ significantly from those used for rapid shallow movements. The former are characterized by slow deformations that can be measured over long periods of time by employing an InSAR-based approach that requires multi-interferogram processing of radar satellite images to detect and quantify movement velocities with an accuracy of 1-2 mm. We show here two examples of slow landslide monitoring using these techniques. Shallow rapid movements, on the other hand, have a sudden onset, thereby entailing completely different techniques and methods. For these types of movements we propose that early warning can be achieved by using an approach combining hydro-meteorological forecasting, soil depth modeling and an infinite slope stability model within a WebGIS system that provides end-users with near real-time information at a basin scale. We describe here case studies that show practical applications of both approaches.

INTRODUCTION
Landslide velocities can span over 10 orders of magnitude (Cruden & Varnes 1996; Varnes 1978), ranging from extremely slow movements of only a few mm/year to extremely rapid catastrophic events that attain velocities of tens of meters per second. While large, slow moving landslides do not usually pose a threat to human life, their continuous or intermittent behavior can cause significant damage to buildings, infrastructure and lifelines, determining costly maintenance and repair works. The large volumes involved also make these events important geomorphologic agents for the landscape. Rapid landslides are generally smaller events, but in the rare cases in which a large, rapid landslide occurs, the results can be catastrophic (Cruden & Varnes 1996; Hungr et al. 2001; Varnes 1978).

Different rates of movement entail completely different approaches and technologies for monitoring and for attempting to forecast the associated hazards. Large slow landslides usually display a continuous or intermittent behavior that makes monitoring possible by means of several different instruments and techniques. These types of movements are set to greatly benefit from modern Earth Observation (EO) techniques and in this paper we describe the application of PSInSAR to two cases of landslide monitoring (Colesanti et al. 2003; Farina et al. 2006; Ferretti et al. 2000). Shallow rapid landslides typically have a sudden onset that is usually not announced by precursory movements, thereby making methods used for long-term monitoring unsuitable. For this reason, most methods focus either on identifying areas more prone to landslides of this type (e.g. Baeza & Corominas 2001), which, however, provide no information regarding when the movements may occur, or on providing early warning by means of weather forecasting (e.g. Aleotti 2004; Keefer et al. 1987). One of the most promising approaches makes use of meteorological modeling based both on short- to medium-term weather forecasts and on the continued development of nowcasting, a technique that produces very short-term rainfall forecasts based on ground weather radar.
observations of the precipitation fields. Coupled with automatic systems that ingest and process this information to produce near-real-time maps of landslide probability of occurrence, this system can provide a few hours of lead time of potentially hazardous events.

**INSAR TECHNIQUES FOR MONITORING LANDSLIDES**

The Permanent Scatterers technique (PS) (Ferretti et al. 2001), developed at the Polytechnic University of Milan, has the capability of determining mm-scale ground displacements using data collected from a SAR satellite. It is based on the identification of a significant number of stable points (i.e. permanent scatterers) on the ground within a radar scene provided that a sufficient number of radar images have been collected. With this method, it is possible to resolve surface motions with a precision of ~0.5 mm/yr while maximum movement velocities vary with satellite revisiting times but are currently limited to 40-50 mm/year, making this technique suitable only for very slow landslides. In addition, as PS usually correspond to buildings, exposed rocks, or other highly reflective artifacts, these have to be present within the landslide perimeter to be able to determine movement rates.

PS can be processed in two different ways. A Regional PS Analysis (RPSA) provides the mean velocity of reflectors located in the investigated area over a given period of time by means of an automatic procedure. This consists in searching for and identifying a linear motion model applicable to each PS in order to extract information regarding linear velocity over the entire acquisition period. The underlying hypothesis is that the displacement trend of a given PS is linear (i.e. remains approximately constant in time). The advantage of this methodology is its rapidity: it is ideal for processing a large number of scenes covering large areas within a restricted period of time. The results of this analysis consist of mean annual velocities for each PS.

An Advanced PS Analysis (APSA) consists in the time series of displacements for every PS and does not assume any linearity in the displacement trend. This approach is more sophisticated, time-consuming and requires skilled personnel. For these reasons, it is suitable for small areas, such as specific high impact landslides or other natural hazards involving slow ground movements, where it is necessary to perform detailed investigations of the phenomenon. The advantages of this analysis consist in a higher density of PS as benchmarks in which abrupt changes in velocity recorded between images are not discarded; in other words, it is not necessary for the PS to have a linear velocity trend. This information, in fact, is often fundamental in landslide investigations. The possibility to observe changes in velocity between image acquisitions allows, for example, to determine precisely when changes in event dynamics occur (i.e. a sudden acceleration in a landslide).

**Regional PS Analysis and Landslide Mapping and Monitoring**

One of the opportunities offered by remote sensing compared to other monitoring methods is the capability of providing synoptic views of large regions. An excellent example of the advantages of EO techniques is illustrated by the results of the European Space Agency (ESA) Service for Landslide Monitoring (SLAM) project (Farina et al. 2006). This initiative had the objective of developing a methodology based on the coupling of information derived from large area PS analyses with the interpretation of optical images and other ancillary data. The use of the ancillary data allows extraction of spatially distributed information from the point-wise PS data and included the interpretation of aerial photos and Very High Resolution (VHR) optical satellite imagery, topographic maps and information regarding land use, geology, etc. The final aim was to provide an innovative landslide risk management tool to
stakeholders. For this reason, a national agency, the Arno River Basin Authority (ABA), was engaged to assess the results and the impact of the methodology on their current practices.

Figure 1: Landslide inventory map of the Arno River basin in Tuscany, Italy. The inset illustrates the detailed information contained in the inventory.

This regional landslide monitoring method was applied to the Arno river catchment in Central Italy. The basin, with a spatial extension of ca. 9000 km², has over 27,000 mapped landslides. A landslide inventory was created by the ABA containing the mass movements classified as active, dormant and inactive (Figure 1) following the terminology proposed by Cruden & Varnes (1996). The inventory, last updated in 2003 and with a reference mapping scale of 1:10,000, was compiled through available references, investigations carried out by local administrations, aerial photo interpretation and field surveys. Aerial photos and VHR satellite images were used to separate landslides from other ground phenomena such as subsidence, swelling and settlement of buildings based on diagnostic morphology. The final product contains attributes related to landslide typology, state of activity, presence of in-situ investigations and information sources.

The contribution of PS to the improvement of landslide inventories regards a better definition of existing movement boundaries, an updating of their state of activity and the detection of previously unknown mass movements. In practice, information regarding the landslide state of activity is gained by overlaying the sparse grid of stable reflectors on the pre-existing landslide inventory map, and analyzing the cases in which the PS benchmarks fall within or near a landslide boundary (Figure 2). A frequent occurrence, in which PS demonstrate their utility, relates to the modification of landslide boundaries. A typical case regards PS in motion that fall outside of a mapped landslide but that are near the boundary; this is a likely indication that the landslide has enlarged since the inventory was made. The velocity of PS within a landslide can provide information regarding the state of activity: no movement indicates an inactive landslide, moving PS indicate an active landslide. Clustered moving PS not located near previously mapped landslides and that are located on a slope can be strong indicators of a new mass movement or of a previously unknown landslide. Used in this way,
PS are useful for checking the completeness and the accuracy of landslide inventories and for their periodic updating.

Figure 2: PS benchmarks and the Arno landslide inventory map overlaid on an orthophoto of a portion of the Arno catchment. Note that some landslides are devoid of PS while others have many. PS are mostly clustered in urban areas.

The set of procedures detailed above can be performed over large areas, making RPSA a useful tool for carrying out landslide studies rapidly, relatively economically and for territories with areas reaching even thousands of square kilometers. However, as shown in Table 1, the number of landslides that actually contain PS information is usually limited. In the Arno catchment, only 6.1% of landslides contained PS information and; while in terms of landslide area, this number grows significantly, it still only covers a small portion of the events present in the basin. When assessing the potential of the technique, however, the following aspects should also be borne in mind: 1) PS are most often located in built-up areas where landslide risk is high and where traditional investigation techniques are limited by the intense modifications to the landscape caused by human activities and; 2) SAR sensors currently being developed and launched will provide significant improvements to these numbers.

Table 1: Landslide and PS statistics for the Arno River basin (see text for more details)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Total landslides in Arno basin</td>
<td>27270</td>
</tr>
<tr>
<td>Landslides with PS information</td>
<td>1664</td>
</tr>
<tr>
<td>% of total landslides with PS</td>
<td>6.1%</td>
</tr>
<tr>
<td>% of landslide area with PS</td>
<td>18.9%</td>
</tr>
<tr>
<td>New landslides detected with PS</td>
<td>223</td>
</tr>
</tbody>
</table>
Figure 3: Descending PS ERS data (1992-2002) overlaid on a VHR satellite image of Cirò Marina. Note that slow movement is present to the east of the discontinuity (magenta line) while none is detected to the west.

Advanced PS Analysis: Two Case Studies
The Advanced PS Analysis (APSA) technique showed its full potential when a sudden ground settlement occurred on 28 July 2004 in the town of Cirò Marina (Italy). The abrupt movement lasted for approximately 5 hours, had a vertical displacement of a few cm and was located along a 3 km fracture running through the town and that to the north becomes an escarpment with a height of up to 2 m (Figure 3). Similar sporadic episodes have been reported in the past in this site. This movement caused severe damage to structures in the urban area but was of unknown origin. Initial hypotheses included: land subsidence induced by fluid extraction (water and/or gas); reactivation of a large landslide and; a seismic creep along a tectonic structure. In order to identify the spatial distribution of the movement connected to the 2004 event and its temporal evolution, i.e. the presence of precursory movements before the event and residual displacements after the failure, an APSA was carried out on all available SAR images (ERS1/ERS2 and Radarsat). In particular, covering the time interval 1992-2002, 60 descending and 30 ascending orbit images were acquired (ERS1/ERS2). Radarsat images were used for the period 2003-2005.

The processed data revealed the presence of slow movements (2-3 mm/y) since 1992 located along a narrow section of the town delimited by the discontinuity and the shoreline (Figure 3). However, the time series of displacements obtained by means of the APSA recorded a sudden movement acceleration between the two SAR acquisitions spanning the event, with average displacements measuring 7-10 mm in the 24 day time interval (Figure 4). The acceleration was not preceded by precursory signs and following the event, residual displacements measured by PS remained significantly higher than the pre-event ones.
Figure 4: Radarsat PS data (period 2003-2004) overlaid on a VHR image. a) Ascending PS: note that all PS movement rates between the discontinuity and the shoreline have similar high values; b) Descending data: note in this case that PS velocities tend to decrease towards the shoreline.

For a correct interpretation of the movement mechanism, it is necessary to accurately characterize the displacement vectors obtained with the PS measurements. By opportune combining both ascending and descending PS data, it is possible to determine both the vertical and horizontal components of the vectors, thereby providing an indication of the actual movement direction. In the case of Cirò, the results indicate that near the linear discontinuity the movements are mostly vertical, while moving towards the shoreline the horizontal component increases noticeably (Figure 4). This information makes it possible to apply a method, consisting in an inversion of the superficial displacement, that provides a reasonable estimate of the depth and shape of the movement slip surface (Carter & Bentley 1985; Cruden 1986). The characterization of the movement geometry is obtained by combining this information with the location of the main scarp. The procedure can be carried out along several different sections and, assuming a rigid deformation between each section, it is possible to produce a 3D reconstruction of the movement (Figure 5). As PS information is only available on land, it is necessary to assume a constant reduction in the local inclination of the failure surface to extrapolate the extension of the slip surface to beneath the seafloor (Figure 5). It is estimated that a movement with this geometry would have a volume of ca. $1 \times 10^9$ m$^3$.

The results of this work are currently being used by the Italian Civil Protection authorities to correctly define the risk scenarios and design appropriate mitigation measures.
Figure 5: (a) Locations of the cross-sections used to estimate the depth of the failure surface; (b) extrapolation to beneath the seafloor by assuming a constant reduction of the local inclination of the slip surface.

The APSA technique can also be used to assess the effectiveness of landslide stabilization works. The Carbonile landslide in Tuscany (Figure 6), a complex movement consisting of earth slides with translational and rotational components was reactivated in 1984 following a period of prolonged rainfall (Farina et al. 2006). Several buildings were damaged and some of these had to be evacuated. To stabilize the landslide, remedial works were carried out, including retaining walls and drainage systems on a portion of the landslide within the town. 310 PS were acquired from ERS1-ERS2 descending data, resulting in a PS spatial density of 114 PS/km², with the benchmark locations clearly correlated to the presence of buildings and other man made structures. An APSA performed on the PS dataset revealed that the stabilization measures were indeed effective as the landslide had stopped moving where the works had been carried out while three other zones, two lateral to Carbonile and one above the town, continue to move with deformation rates of up to 12 mm/year. In addition, the PS analysis also revealed that another portion of the town previously considered stable, was in fact sliding at a rate of ca. 5 mm per year. This information led to the modification of the landslide perimeter as shown in Figure 6.
Figure 6: Aerial photo of the Carbonile landslide in Tuscany. The landslide boundary was changed following the successful stabilization of a portion of the landslide. Red (dark) line = previous landslide boundary; yellow (light) line = new landslide boundary; green (light) dots = immobile PS benchmarks; red (dark) dots = moving PS benchmarks.

EARLY WARNING OF RAPID SHALLOW LANDSLIDES
Soil slips generally occur following intense storm rainfall and often transform into debris flows. Although they are usually modest in size at initiation, they are able to efficiently erode loose material from the hillslope and scour channels, and thereby rapidly increase in volume. Combined with their high velocities and the size of the debris transported, these events can be very hazardous for human life, property and infrastructure. Furthermore they do not provide precursory signs such as ground deformation or slow movements prior to initiation. These characteristics make the techniques described above for monitoring slow-moving landslides unsuitable; to date, meteorological forecasting is the only viable technique for early warning.

PREVIEW, a project developed within the European Sixth Framework Programme (FP6), was dedicated in part to the development of a method based on the combination of advanced hydrometeorological forecasting, geotechnical modeling and WebGIS visualization techniques for producing semi-automated tools for monitoring and early warning at basin scale. Traditional weather forecasts such as the limited area model (LAMI) employed in Italy can provide a first level of early warning with a lead time of 72 hours. Where ground-based weather radars are present, it is possible to use observations of the rainfall field associated with extreme weather events to predict rainfall evolution in the very short-term. This
technique, known as rainfall nowcasting, appropriately downscaled to 1 km x 1 km ground resolution grids, can provide an effective means for forecasting potential landslide generating events with a lead time of up to a few hours.

Figure 7: Basic structure of the data flow for the shallow landslide early warning system being developed within the EU FP6 PREVIEW project

The system being developed at the Earth Sciences Dept. of the University of Florence (UNIFI) combines these meteorological modeling techniques (provided by CIMA – Centro Interuniversitario per il Monitoraggio Ambientale of the University of Genova) with an infinite slope limit equilibrium stability model and advanced WebGIS (currently hosted by Telespazio Srl) tools. The operational chain is completely automated: the weather data is continuously downloaded via FTP to UNIFI which processes the data in near-real-time and posts it to the WebGIS server (Figure 7). Updates are ordinarily posted at 6 hours intervals but, when the first level of alert is exceeded, new data is posted every 3 hours. Local stakeholders can access and download the data at any time. The main goals are to identify and provide early warning regarding areas with the highest potential for the occurrence of shallow landslides at basin scale and to develop a tool for rapidly activating civil protection resources in cases of impending extreme events.

The rainfall data is used to generate soil saturation maps which are then used as input for the infinite slope stability model of Skempton & Delory (1957) which produces a distributed factor of safety (FS) for every pixel of the DEM. In addition to soil saturation, the method requires data regarding slope angle, soil depth, material unit weight, cohesion and angle of internal friction. The use of an innovative soil depth model developed by Catani et al. (in preparation) for estimating the thickness of the colluvial layer within the basin represents another strong point of the model. This soil depth model has already been applied and validated in several basins and has given promising results: mean errors in soil depth vary between 11 and 40 cm compared to much higher values provided by other methods. As the important role of soil depth in assessing slope stability in soil mantled landscapes is well-known (e.g. Johnson & Sitar 1990), the use of this model significantly increases the accuracy of the approach. The output of the slope stability algorithm is a factor of safety distributed over the entire basin. This data is transferred to a WebGIS system which enables users to browse, select, and download all information for further processing (Figure 8).
Although the system is still a prototype, it has been fully functional for several months. Further developments regarding several different aspects such as the improvement of the soil infiltration scheme, will soon be coupled directly with the slope stability algorithm based on an approach similar to the one proposed by Iverson (2000). The use of ensemble rainfall forecasts and the introduction of uncertainty in the input parameters produce a probabilistic factor of safety, and continuous enhancement of the WebGIS interface.

![Figure 8: Detail of the WebGIS interface of the shallow landslide early warning system](image)

CONCLUSIONS

We have shown here several case studies that demonstrate the different techniques and methods that can be used for studying both slow and rapid landslides. Slow generally deep-seated movements can be effectively monitored by InSAR remote sensing techniques. Over large areas, even thousands of square kilometers, it is possible to use a Standard PS Analysis to monitor a large number of landslides and to reliably update features contained in landslide inventories such as movement states of activity and boundaries and even to detect previously unknown landslides. The main limitations of the technique are the necessity to have stable reflectors (e.g. rock outcrops, bare soil, buildings, roads, etc.) located on or near the landslide and the possibility of monitoring only extremely slow movements.

The sudden ground movement of Cirò Marina illustrated the application of an Advanced PS Analysis. The time series of the ground displacements allowed the changes in behavior of the movement to be highlighted and, more importantly, the analysis of both ascending and descending data allowed the movement geometry to be determined by means of a simple graphical method. This information was sufficient to accurately characterize the movement in terms of velocity, geometry and volume. The drawbacks of APSAs are the same as for the Standard Analysis with the addition of the higher costs required for the data processing. All
PS methods however will benefit significantly from the continuing advances in the development and refinement of the processing algorithms and from the launch of new generations of sensors with lower revisiting times and more advanced radar instrumentation.

The absence of precursory signs makes the monitoring of shallow rapid landslides with the instruments used for slow movements impossible. It is therefore necessary to adopt completely different approaches based on advanced weather forecasting techniques and to couple these with geotechnical modeling and data transfer and visualization technologies capable of providing information to stakeholders rapidly and effectively. The system proposed here organizes different advanced approaches, such as hydrometeorological forecasting techniques and WebGIS visualization tools, and more standard components, such as the infinite slope limit equilibrium stability approach, although significantly improved by the coupling with an innovative distributed soil depth model, to produce a sophisticated yet practical tool capable of providing near-real-time warning of shallow landslide occurrence to civil protection authorities.

REFERENCES


initiation.” Canadian Geotechnical Journal, 27(6), 789-801.

ACKNOWLEDGEMENTS
The authors wish to thank the numerous parties that contributed to the work described in this paper. These include: the Italian Department of Civil Protection, the Arno River Basin Authority, Telerilevamento Europa, Telespazio, Roberto Rudari and Simone Gabellani of CIMA and all members of the Engineering Geology Group of the Earth Sciences Dept. of the University of Florence. In particular, we acknowledge Sandro Moretti, Filippo Catani, Gaia Righini, Paolo Farina, Angelo Benedetti and Samuele Segoni for their important contribution.
ASSESSMENT OF SLOPE SENSOR DATA TO SUPPORT ROCK SLOPE STABILITY ANALYSIS AND INFRASTRUCTURE HAZARD MANAGEMENT

D Jean Hutchinson, Mark Diederichs, Kathy Kalenchuk and Matt Lato
Department of Geological Engineering, Queen’s University
GeoEngineering Centre, Queen’s University and the Royal Military College
Kingston, Ontario, Canada

Abstract: Two case histories are presented to demonstrate the use of sensor data in an integrated assessment process, including GIS based process models and numerical simulation to evaluate the potential for further slope instability. The first case, a large, active, slowly moving slide located above a hydro power reservoir provides a very rich history of data from conventional geotechnical sensors, and interpretation over several decades of monitoring and drainage. Ongoing work is focused on developing a calibrated three dimensional model to assess the effects of probable changes in the future. The geometry of the shear zones within the landslide is being re-evaluated using geospatial statistics, and will be considered during the geomechanical modeling process. The second case discusses rockfall hazard along transportation corridors. Here, risk is managed by regular inspection and classification of the slope, and mitigation when warranted. In this case, LiDAR, a remote sensor, has been applied to collect information about the geometry and structure of the rockmass. The process developed and applied to date provides a detailed assessment of the volume and size of the blocks of rock that could be generated during slope failure. Inclusion of this data into GIS based analysis of rockfall hazards will result in refined hazard mapping.

INTRODUCTION
Integration of geotechnical sensor data with geological and geometric data, coupled with geomechanics process modelling, provides an effective way of assessing landslide behaviour and movement trends for active slopes. Geological data may be collected from physical sensors and tests, including drill hole logs, survey monuments, inclinometers, extensometers and piezometers. Remotely sensed data, from tools such as LiDAR or 3D photogrammetry, may also be used to derive geological and rockmass classification data.

The effective combination of this data to support risk decision making is at the heart of the Geotechnical Insitu Sensor Technology, or GIST, project (Figure 1). Participants in this project include researchers at Queen’s, Alberta, Laval, and Windsor Universities, and industrial partners include the two national Canadian Railway companies, CN and CP, Transport Canada and BC Hydro. The research findings from this project are being applied to rail hazards in the Railway Ground Hazard Research Project, which has most of the same research and industrial partners. As a result of this diverse set of partners, the project has generated slope management case histories ranging from discrete, large, densely instrumented, slopes with a wide array of physical sensors and a well-established knowledge base, through to spatially distributed hazards such as rockfalls along railway corridors. This paper discusses the geomechanical evaluation and risk management context of two of the cases developed during this project.
This paper presents two case histories, providing the opportunity to contrast hazard evaluation approaches used for each. The first, a large, active, slowly moving landslide is located above a hydro power reservoir, in a large, previously glaciated valley. The slide involves a volume of $1.5 \times 10^9$ m$^3$, and has been monitored for over 30 years. The hazard is controlled by drainage of the landslide, which has been in place for over 25 years. The objective of the current work on this site is to assess the influence of the range of possible interpretations of the shear zone on the modelled behaviour, and to select the most representative model(s) for use to assess the potential for extreme, but reasonable events that could accelerate the slide movement. A GIS model has been built to display and interpret the geological and sensor data (Kjelland et al. 2004a; Kalenchuk et al. 2008). The displacements recorded on ground surface and on the lower shear zone indicate movements that vary both in sliding direction and in magnitude, creating a truly three dimensional map of displacement vectors. Interpretation of the available data indicates a basal shear zone that is up to 245m below ground surface with variable thickness. The position of the shear zone is established by core logging from numerous boreholes, but will never be known with certainty between the boreholes. The shear zone position has been interpreted from the known data, considering the geological history of the slide, using a variety of statistical approaches. The quality of the data fit is discussed.

The second case history considers rockfall hazard along transportation corridors. Generally, in these cases, limited data is available about the frequency or magnitude of the rockfall hazard. In an effort to quantify the hazard, geological mapping, geotechnical scanline mapping, and mobile LiDAR data has been collected, at a number of locations along highways and railways. The LiDAR data has been used to develop a detailed three dimensional image of the rockface surface, to quantify the orientation of structural surfaces. This provides data for an evaluation of rockfall initiation potential, block size and distribution, and process modelling of the failure mechanism, to provide an assessment of the rockfall potential and to provide a comparison between the effectiveness of these data sources.
LARGE, SLOWLY MOVING SLIDE ABOVE A HYDRO POWER RESERVOIR
The Downie Slide is located 64 km north of the Revelstoke Dam on the Columbia River in interior British Columbia, Canada (Figure 2). The rock slide measures 3.3 km from toe to head scarp, 2.4 km along the valley and has a maximum thickness of 245 m, as shown in Figure 3, producing a rockslide of a volume of $1.5 \times 10^9$ m$^3$ (Enegren & Imrie 1996). The slide is characterized by an irregular head scarp, with maximum height of 150 to 180 m. The surface of the slope generally dips at 18° into the valley, except near the toe where it dips at 40° (Imrie et al. 1992).

The slide is thought to have been initiated just after deglaciation, approximately 9,000 to 10,000 years ago, when the pore pressures within the valley walls were still very high. The total movement of the sliding mass is approximately 250 to 300m, but it is hypothesized that none of this movement has been rapid (Imrie et al. 1992).

Geological Setting
The bedrock stratigraphy consists of high-grade metamorphic interlayered and well-foliated mica gneisses, mica schists and quartzites, belonging to the Shuswap Metamorphic Complex. The rock within the sliding mass is moderately blocky and seamy, with alternating bands ranging from 0.1 m to over 10 m thick. The foliation generally dips easterly towards the river at an angle of 18 degrees, but is locally disrupted by intricate complex folds and small-scale faults (Imrie et al. 1992; Imrie 1983; Lewis & Moore 1988; Moore 1989; Piteau et al. 1978). The gneisses and quartzites are hard and brittle, whereas the mica schist is soft and yielding. Sheared seams containing gouge (1 mm to 150 mm thick) are common, joints that cross cut the foliation are widely spaced, and the schist is highly erodible when sheared or when exposed to flowing water (Lewis & Moore 1988). The base of the slide is defined by a large continuous sheared and brecciated Lower Shear Zone reported to be 15 m to 20 m thick (Imrie et al. 1992). New estimates, based on interpretation of borehole data, indicate that the Lower Shear Zone thickness varies between 1.2 and 62.5m, and that a middle and upper shear observed in the core may be contributing to the displacements measured by the inclinometers (Kalenchuk et al. 2007). The shear zones are sub-parallel to both the foliation and ground surface and contain seams of plastic silty clay gouge.

Investigation of the Downie Slide
Discovered before the Revelstoke reservoir was impounded, the slide has been under active investigation and monitoring by BC Hydro since the 1970’s. While an observational approach might have been adopted at that time, relying on instrumentation data to record landslide movement and provide warning, the Vajont dam failure was still unexplained at that time (Imrie et al. 1992). As a result, a drainage system, comprising 2430 km of adits, and 13,600 km of drainholes, was installed in the late 1970’s and early 1980’s. The drainage system was designed to reduce pore pressures within the slide mass sufficiently to offset the potentially destabilizing effect of the slide toe impoundment by the reservoir (Enegren & Imrie 1996), expected to lead to a reduction in factor of safety against sliding by 7%, if drainage was not installed (Imrie et al. 1992). Furthermore, a dense array of instruments was installed into the boreholes drilled during several site investigation and instrumentation programs carried out at the site, as described by Imrie et al. (1992) and Enegren & Imrie (1996). Data has been collected for over 25 years, allowing an examination of the rate and direction of slide movement, and changes in piezometric pressure, to be made. The rate of slide displacement was reduced from a pre-drainage average rate of 10mm/year in 1975, to approximately 2mm/year in 1983. The piezometric pressures at the base of the slide were lowered by as much as 100m, even though 70m of the toe was inundated (Imrie et al. 1992). Slowly
increasing displacements in the subsequent years have been attributed to biological clogging and offset of the drainage holes over time. A project to replace some of the drain and instrumentation holes was carried out during the early 1990s (Enegren & Imrie 1996)

Figure 2: Downie Slide location (from Kjelland 2004; after Imrie 1983)

Figure 3: (a) Airphoto of the Downie Slide, and (b) East-West cross-section taken looking north through the centre of the slide
Two Dimensional Numerical Simulations

Numerical simulation of the slope deformation has been carried out, and determinations of factor of safety against failure have been made (Imrie et al. 1992, Moore et al. 1997, Kjelland et al. 2004b; Diederichs 2007). To date, these evaluations have been made on a number of representative two dimensional sections through the sliding mass. Modelling conducted by BC Hydro was based on the assumption that this active, slowly moving landslide was at a factor of safety of 1 under conditions of high water pressure, normally encountered during the spring, allowing back analysis to determine a representative friction angle. Using this calibrated model, it was calculated that the drainage system, as installed, improved the factor of safety by up to 14.5% (Imrie et al. 1992). Subsequent analysis of the stability was completed in 1994, because some of the piezometric levels had risen again to the levels observed before drainage was installed. These calculations indicated that the improvement in the factor of safety compared to pre-drainage conditions was 7 to 11% (Enegren & Imrie 1996).

Kjelland (2004a; 2004b) carried out a number of numerical simulations of the Downie Slide, using a model calibrated to the instrumentation data. This work explored the appropriate strength parameters and models for the analysis of the sliding mass. Figure 4 presents a plot of the factor of safety of the original slope depending upon different combinations of cohesion and friction angle.

![Figure 4](image_url)

Figure 4: Factor of safety calculated for reasonable ranges of friction angle and cohesion. The black line shows combinations for factor of safety of unity. Modelling was completed using SLIDE, a 2D limit equilibrium program, for a cross-section through the centre of the slide, and for conditions prior to installation of the drainage system (after Kjelland 2004).
Kjelland (2004a, 2004b) conducted further evaluation to assess the effect of different failure criteria on the development of the lower shear zone, following deglaciation (Figure 5). The water table positions used in these models were those assumed to represent the post-glaciation period, and then the shear zone was allowed to develop. The quality of the model was assessed by the position of the shear zone, and the thickness of the shear band that developed. The results of this work also include simulated inclinometer output as shown in Figure 5. The ubiquitous joint model shows the best fit for both assessment parameters.

Probabilistic analysis was conducted to demonstrate several concepts (Diederichs 2007). This work was completed using the SLIDE program, on a two dimensional section through the middle of the slide, and considered the effect of drainage, and the effect of reasonable variation in the slide surface position. It should be noted that the probability of failure determined from the assessments shown in Figure 6 should not be taken as absolute, as the model is intended to be illustrative, and therefore was not rigorously calibrated.
(a) Hypothetical worst case scenario - Pre-drainage, full reservoir. 
This condition was never allowed to develop, as drainage was installed several years before the reservoir was impounded.

(b) Post-drainage, with largest draw-down achieved, full reservoir

(c) Variation in shear surface position - Post-drainage, with largest draw-down, full reservoir

(d) Post-drainage, with some loss of drainage effectiveness, full reservoir

Figure 6: Numerical simulation of a 2D cross-section through the centre of the slide, to assess the effect of water table and shear surface location variation, completed using the SLIDE program (Diederichs 2007). Results are provided for an illustrative comparison, and should not be considered to be representative of true conditions.

Comparison between the upper two sets of images in Figure 6 shows the dramatic reduction in probability of sliding of the landslide, achieved when the drainage system was fully operational, soon after installation and before reservoir impoundment. It must be noted, though, that the upper diagrams are for a condition that never existed at the site, of full reservoir impoundment and no drainage of the landslide mass. Furthermore, it can be seen
that the probability of sliding increases slightly with either some uncertainty in the position of the lower shear zone, or with a rebound in the water table position due to a reduction in the effectiveness of the drainage system.

Re-interpretation of the Shear Zone Location
The current study, discussed by Kalenchuk et al. (2007; 2008), is focused on developing a three-dimensional model, considering the effect of geologically-plausible variation in the position and extent of the shear zones found within the slide mass (Figure 7), and the variation in piezometric pressure that might be expected in the future. This objective of this work is to evaluate the effect of the position and geometry of the shear zone on the numerical simulation of the movement of the sliding mass. Further simulations, using the calibrated model, will also be used to assess the influence of reduction in drainage system efficiency and the potential change in the climate over time.

Figure 7: Geometry of the Downie Slide and Shear Zones (after Kalenchuk 2007)
Interpretation of the shear zone position is based on known intersections from the borehole logs. The borehole locations are shown in Figure 7, relative to the outer boundaries of the landslide. Measurements of the shear zone depths (Figures 7c, d, e and g), and Lower Shear Zone thickness (Figure 7f), found from the borehole data, are also shown in this figure. Many previous interpretations have largely assumed a straight line interpolation between known points. The work completed to date by Kalenchuk is focused on developing geologically-reasonable surfaces for the middle and lower shear zones, as shown for the Lower Shear Zone in Figures 7d and e. The positions of the known points are respected, and several spatial statistical methods have been used to grid the geometric data, as shown by Kalenchuk et al. (2007).

Gridding is a process that uses interpolation and extrapolation algorithms to create a surface by defining the elevation at regularly spaced (x, y) grid points based on the known elevation at irregularly spaced sample locations. Each gridding method uses a different mathematical algorithm to compute the weighting assigned to sample data during the interpolation of gridpoint values at un-sampled locations. In order to compare the usefulness of various algorithms, and to select an appropriate interpolation method for the available data, a cross-validation process is used as a basis for comparison. Cross-validation is achieved by removing one data sample at a time from the data set and determining the interpolated value for each point based on the remaining data in the set. This returns a series of error values defined as the difference between the true elevation value and the interpolated elevation value at each sample location. Evaluation of this data for the Downie Slide case history shows that the best results are generated by using kriging (Figure 7d) and multi-quadratic functions, although there are still some fairly significant cross-validation errors around the outer sides of the slide mass, where limited sub-surface data is available.

The definition of the position of the Lower Shear Zone in the upper part of the landslide requires some attention. The position of the head scarp in plan is well defined from mapping and aerial photography, with a height of between 150 and 180m. However, the connection between the scarp and the underlying shear zones is not known, as there is no additional information about the shear zone position in this region. At the two extremes of possibility, the shear zone may connect to the scarp at ground surface, or it may extend downward parallel to the head scarp orientation, to meet the shear zone along a relatively smooth surface defined by the known points lower down in the slope. These two possibilities are shown in cross-section in Figure 8.

![Figure 8: Possible limits of interpretation of the position of the Lower Shear Zone in the upper part of the Downie Slide (from Kalenchuk et al. 2007)](image-url)
The selection of a single shear surface geometry is somewhat subjective. As a result, during later stages of this research, numerical modeling will be used to assess the impact that large- and small-scale geometric variability has on the simulation of the overall landslide behaviour. Parameters of interest during this simulation series include the development of representative displacement along the shear zones, and ability of the model to replicate the three-dimensional nature of the downslope movement, as shown in Figure 9, both in terms of direction and magnitude. Figure 9 shows the data collected during the period after reservoir impoundment, while the drainage system was still at maximum effectiveness. Similar plots have been produced for the displacements recorded at the Middle and Lower Shear Zones, to allow for detailed analysis of the modeling results.

![Figure 9: Ground surface movement from survey monument data (Kalenchuk 2007)](image)

**Re-interpretation of Piezometric Pressures**

The monitoring data includes piezometric pressure from a number of levels within a number of boreholes. The positions of the boreholes, in plan view, are shown with open red circles on Figure 7. Interpretation of this data includes consideration of whether the shear zones contain sufficient gouge and are of sufficient thickness to create a hydraulic barrier to vertical flow within the landslide mass, and therefore whether zones with different pore pressures could develop.

At the time of the drainage system installation, the piezometric pressure within the slide rockmass was assumed to be continuous. The gouge filled shear zones could have created potential hydraulic barriers. However, reaction tests, where flow from some of the drainage holes was blocked, generated a response in the piezometric pressure measured 600m away. It was therefore suggested that the gouge filled shear zones had been breached by the substantial slide movement over time, thereby introducing higher conductivity windows, and preventing the development of perched water (Imrie et al. 1992).

By comparing piezometric data for tips located on opposing sides of the shear zones, it can be determined whether multiple water tables exist, and if they are confined by low permeability shear surfaces. Three water tables have been identified in the vicinity of Downie landslide, one below the lower shear, one between the lower and mid shear, and one above the mid shear as shown in Figure 10. There does not seem to be any distinction between the water tables located above or below the Upper Shear, thereby indicating that the upper shear is permeable. This is consistent with the observations made by Imrie et al. (1992). However, as
can be seen in the cross-section presented in Figure 10d, there appears to be some variation in the piezometric pressures within the slide mass, affected by the Middle and Lower Shear Zones. The influence of these pore pressure variations will be assessed as the numerical simulation work proceeds.

**Infrastructure Risk Management**

The objective of the drainage works installed at Downie Slide as stated by Imrie et al. (1992) was to “achieve an increase in stability which would more than offset the decrease in stability caused by submergence of the slide’s toe”, and to “install a comprehensive monitoring system at Downie to evaluate the effects of such remedial measures”. In this case, regular monitoring, observation and assessment of the data recorded has resulted in the remediation of the drainage adits, and several projects to install additional boreholes and drainholes.

The philosophical justification for this approach was based on the assumption that the slide had been moving for over 9000 years, and therefore could be considered an active, but slowly moving landslide, that had undergone a series of fluctuations in piezometric pressure and earthquake loading over that time, but was still largely stable. The times of accelerated movement could be correlated to high piezometric pressure, and therefore the sliding mass was at a factor of safety of 1 during these times. Any reduction in the piezometric pressure by drainage, that more than offset the increase in pressure created by reservoir filling would not make conditions worse, and therefore was considered to be an improvement on pre-filling conditions.

![Piezometric pressures measured above the Middle Shear Zone](image1)

![Piezometric pressures measured between the Middle and Lower Shear Zones](image2)

![Piezometric pressures measured below the Lower Shear Zone](image3)

![Cross-section through the centre of the landslide showing interpreted positions of the water tables associated with the area above the Middle Shear Zone, between the Middle and Lower Shear Zones, and below the Lower Shear Zone](image4)

**Figure 10:** Measurements of piezometric pressure within and below the sliding mass (after Kalenchuk 2007)
The probability of landslide acceleration has been re-evaluated on a regular basis throughout the life of the monitoring project. The observational approach to risk management requires regular re-assessment of the design assumptions and the conditions present within the slide mass. As long as the slide stability conditions are managed so as to remain better than at the start of engineering works at the site, the risk management process as defined here will continue to be deemed a success. It is interesting to note that Imrie et al. (1992) state that the observational approach, utilizing instruments to monitor the response to controlled reservoir filling, would likely have been implemented for this huge, but only moderately dipping mass, if the Vajont failure had not recently occurred, at the time of evaluation of the Downie Slide. Imrie et al. (1992) note that the implementation of the drainage system was considered to be a conservative approach, which was appropriate at that time.

**INTERMITTENT ROCKFALL ALONG TRANSPORTATION CORRIDORS**

Rockfalls are relatively regular events along transportation corridors adjacent to rock slopes, and are generated both by natural rockfall events, and from rock cut slopes. The rock may land on or damage the track or highway and may result in accidents. There is generally a need to close the corridor while the material is cleared away, and the slope is stabilized, if required.

Due to the substantial extent of possible rockfall source zones adjacent to transportation corridors in hilly or mountainous terrain, it is generally not technically possible nor financially feasible to mitigate all of the hazards at the source or to protect all sections of the road or track. Therefore, risk assessment is carried out to assess the hazards and to focus mitigation efforts on the most hazardous areas. Furthermore, the risk is managed by regular inspection and review of the rock slopes, and in some locations by providing a warning. In the case of highways, vehicles are able to move laterally, within the confines of the road bed, and are generally able to stop in a reasonably short distance, if the hazard is sighted. Warning is generally provided by signs which indicate the potential for rockfall hazard. On the other hand, trains on track are confined to the rail bed, with no possibility to move around a hazard. Generally trains have fairly large stopping distances, depending upon the speed, grade of the track and visibility of the hazard. Warning can be provided by operational instructions, including mandating slower operating speeds for certain sections of track, and by track-side warning devices.

In the work reported here, the structure and geometry of potential rockfall source zones is being assessed using remotely sensed geometric data, as shown in Figure 11. Such data is increasingly being collected for use in geotechnical assessment of slope hazards, ranging from very large scale cases, at the scale of mountain sides (Sturzenegger et al. 2007; Strouth & Eberhardt 2006), through to assessment of rock masses exposed at outcrop or rock cut scales (Lato et al. 2007a). Detailed geometric data may be acquired from photogrammetric analysis, or from LiDAR surveys, and results in a three-dimensional view of the object scanned. The resolution and quality of the survey depends upon the scanning equipment used, the distance from the object and the type of software used to process the data.
Good resolution data is very useful for assessing the orientation of rock mass structures, and the size and shape of blocks defined by these structures. Very high resolution data may be used to assess the roughness and condition of individual joint surfaces (Haneberg 2007).

Collection of LiDAR data by the authors has been carried out at a number of sites along linear infrastructure corridors, ranging from highway rock cuts to railway tracks traversing natural slopes. Previous work by other members of the research project (Martin et al. 2006) has focused on using airborne LiDAR data to identify the location of potential rockfall source zones along steep, natural cliffs adjacent to rail track, and to assess the risk to the railway tracks below.

The focus of the project reported here is to develop a technically viable, robust, and simple workflow to define the process from LiDAR data collection through to rockfall analysis. This includes considerations of the design of the survey, the resolution and rate of data acquisition provided by the equipment, and the objectives of the data collection and analysis required. To evaluate the various LiDAR scanning systems and the quality and usefulness of the resulting data, a survey test site was established. Located near Sunbury, Ontario, the road cut measures approximately 5m in height, as shown in Figure 12a. This cut does not pose a rockfall hazard to the highway, because it is offset from the road by a ditch, and because loose rock was removed during construction by scaling. This site is being used because the rock structure is visually very clear and distinct, and access to the rock cut for conventional mapping is
available, providing an ideal setting for testing both the equipment operation and the quality of the interpreted structural data.

LiDAR surveys of the Sunbury rock cut were acquired using three different systems, as noted in Figure 12. The large feature, dipping to the right hand side of Figure 12a, can be seen in all of these images. Images 11b through 11d are ordered in sequence from most rapid acquisition rate to slowest. The lowest resolution, TITAN data, shown in Figure 12b, is close to the minimum density of data required for identification and rigorous analysis of the structural orientations.

Technical parameters useful for the selection of an appropriate survey method are provided in the captions on Figure 12, including the time required for data acquisition and the resolution of the data at the relatively short distance over which the surveys were taken. This information is appropriate for data collection over a linear section of a transportation corridor. Where the rock face to be evaluated has a non-linear shape, such as where it has been exposed in a curved rock cut, the collection of data is most easily completed using a mobile, terrestrial system. An example of this type of data, taken by the TITAN system, is shown in Figure 11.

A comparison between the structural data interpreted from the LiDAR data and collected by conventional mapping is discussed by Lato et al. (2007a). Data processing of the very detailed three dimensional surfaces produced from LiDAR surveys depends upon the development of a reasonably representative triangular mesh overlaid on the raw data. The mesh can be developed such that individual triangles include a huge or very small number of points, thereby ranging from very few triangles to a very large number of triangles. The connection of the triangles into contiguous surfaces requires definition of the allowable range of triangle orientations that are grouped together during the processing. In the case shown here, the vectors orthogonal to the triangles have a total range of 8 degrees in variation. Discussed in more detail by Lato et al. (2007a), this process leads to the types of stereonets shown in Figure 13. The optimum balance for number of points per grid cell, and number of triangles per discontinuity, and therefore the most representative stereonet, is found at the inflection point in the curve shown on this diagram.

The road cut structure interpreted from the LiDAR data was compared to the results of geomechanics mapping at the site. The results of this work (Figure 14), showed that there were differences in the scatter in the data collected, and in the orientations of the structure interpreted. Appropriate survey design requires more than one survey set-up point, to acquire data from orthogonally oriented structures, and therefore to reduce the potential for horizontal occlusion of data. This is particularly important for a rockmass where minimal relief is created by the exposed structure. In this case, it is thought that the differences between the two data sets could be due to a number of factors, including difficulties with co-registering the orientations of the two surveys, as detailed location surveys were not acquired during the LiDAR survey, and due to the influence of regional deviation in the magnetic declination.
(a) Photograph, showing the roadcut outcrop and road surface. View is towards the east.

(b) Mobile terrestrial scan using Terrapoint Titan system mounted on a truck. This is a time of flight sensor. View is towards the north-east. Time for data acquisition is 5 to 10 minutes per direction of traverse. Both directions are surveyed. GPS base station is required to be functional for 4 hours. Data resolution is 400 points/m².

(c) Stationary terrestrial scan using Leica Geosystems HDS 6000. This is a phase based sensor. View is east/south-east. Image is created from three separate scans at different positions. The data shown is sub-sampled at ½ ratio. Time for data acquisition is 10 minutes for each set up location. Data resolution is 40,000 points/m², at medium density acquisition level.

(d) Stationary terrestrial scan using Optech ILRIS 3D. This is a time of flight sensor. View is to the east. Image is created from three separate scans at different positions. Time for data acquisition is 45 minutes per set up location. At least two set up locations are required. Data resolution is 5,000 points/m².

Figure 12: Test site, at a road cut near Sunbury, Ontario, Canada, from Lato (2007). North is to the left in (a)
Figure 13: Influence of data processing on structural data interpreted from LiDAR data (from Lato et al. 2007a). Numbers superimposed on the stereonets indicate the value of the largest contour, and in brackets, the contour interval.

Figure 14: Stereonets of structural data collected at the Sunbury test site, by (a) conventional geomechanical mapping, and (b) interpretation of three merged LiDAR scans (from Lato et al. 2007a).

Kalenchuk et al. (2006) developed a new method for examining and evaluating the geometry of a rockmass. Based on structural data measurements, the process involves creating a 3DEC model to replicate the joint orientation and variability. The geometry of the blocks so generated is then evaluated to consider their size and shape distributions, as shown in Figure 15. The logic for describing the block shapes is based on an evaluation of the relative length and geometry of the sides, leading to definition of cubic, elongated or platy shapes. Further information about this process can be found in Kalenchuk et al. (2006).
The block shape distribution can be plotted on a ternary diagram, as shown in Figure 15c. Here, the characterization of the shape can be evaluated. The data presented in this figure should be related to Figure 15a and 15b. For example, Figure 15b shows that the $D_{40}$ fraction comprises blocks that are smaller than $1 \text{ m}^3$ in volume. Figure 15c shows that the largest blocks (greater than approximately 4 to 5 m$^3$), are cubic to cubic-elongated. A substantial number of blocks are elongated or platy, but these blocks are all of small volume.

![Diagram of block distribution](image)

(a) 3DEC model of rockmass

(b) Evaluation of block shape and volume from 3DEC model

(c) Ternary diagram showing classification of block shapes and distribution of sizes

(d) Block volume distribution

Figure 15: Evaluation of block volume and size distribution for the Sunbury test site outcrop (from Lato 2007) based on methodology developed by Kalenchuk et al. (2006)

The analysis of the Sunbury test site data has shown that a much more rigorous approach to rockmass characterization is possible when detailed geometric information about an outcrop is available. This data can be used in rockfall modeling to identify both the source zones (airborne survey) and the rockmass structure (airborne combined with terrestrial). This will make the development of volume-specific rockfall assessment possible. Furthermore, once rockfall modeling programs capable of modeling different rock shapes are available, this process will supply the required data.

**Infrastructure Risk Management**

Assessment of rockfall hazard along linear infrastructure corridors is generally based on a defined rock mass assessment approach. Most methods used by railway or highway operators in Canada are based on two main components, including: i) classification of the geometry of the site, including the height and slope angle of the rock cut or natural slope, as well as the configuration of any ditches, and ii) assessment of the geological conditions, largely based on
structural orientation, and in some cases on an assessment of block volume. The ratings that result from this work are used to prioritize mitigation efforts along the transportation corridor. Repeated assessments of conditions over time are intended to focus mitigation efforts on unstable areas that have developed.

As noted previously, for railway operations, when the rockfall source zone cannot be mitigated, or when it is not technically feasible or financially viable to protect the track, track operation protocols, including daily track inspection, reduced operating speeds, and the installation of hazard warning systems, help to manage the risk.

With repeated surveys of the slope, it is possible to examine the difference between the slope geometry at different times, thereby allowing for location of the source and accumulation zones, and calculation of failed volumes. Rosser et al. (2005) have demonstrated this from data collected during a regular survey of a coastal cliff.

CONCLUSIONS
The assessment of slope stability using conventional sensors and remote sensors has been discussed in this paper. In both cases, that data collected from the sensors provides the basis for understanding of the hazard posed by the mass movement, and is integrated into a process whereby numerical simulation is used to assess the potential for future changes in the slope condition. GIS is a very useful platform for assessing large amounts of spatially related data. LiDAR and other remote sensing approaches provide mapping and geometrical data from sites which are difficult or impossible to reach to carry out conventional surveys.

REFERENCES


ACKNOWLEDGEMENTS
The comments from the Technical Committee of the Forum are gratefully acknowledged.

The research projects reported could not have been completed without the research funding provided by GEOIDE, NSERC and Transport Canada, or without the site and data access provided by BC Hydro and CN Rail. Thanks are due to these groups for their support of this work.

Substantial technical contributions to this work have been made by a number of students and collaborators on the GEOIDE and RGHRP projects. In particular, Rob Harrap, Queen’s University, Neil Kjelland, AMEC Consultants, Dr. Tim Keegan, Bruce Geotechnical Consultants, Dr. Martin Lawrence, BC Hydro, and Dr. Carlos Carranza-Torres, have contributed very significantly to the research work reported in this paper.
DEVELOPMENT AND APPLICATION OF GEOINFORMATICS FOR LANDSLIDE RISK MANAGEMENT IN HONG KONG

K. C. Ng and H. N. Wong
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Abstract: Since the early 1990s, the Geotechnical Engineering Office (GEO) has been taking advantages of advances in information and related digital technologies in carrying out geotechnical work. Significant efforts have been made by the GEO over the years in developing key geoinformatics datasets, including acquisition of new data in digital format and digitization of old records, particularly those of territory-wide coverage. These have paid valuable dividends as digital technology is playing an increasingly important role in landslide risk management in Hong Kong, ranging from data management and dissemination, analysis and interpretation of geospatial data, to providing critical information for problem solving and decision-making. Some of the development work of geoinformatics at the GEO are recognized as the state-of-the-art that have helped to advance geotechnical practice and to enhance the capability and efficiency of conducting geotechnical work in Hong Kong. Furthermore, applying novel digital technologies has revolutionised the way in which landslide risk management is conducted.

INTRODUCTION
Since the early 1990s, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) of the Government of the Hong Kong Special Administrative Region (HKSARG) has been taking the lead in exploring the potential of applying novel digital technologies to carrying out geotechnical work (Wong 2004a, b). Considerable efforts have also been made in the compilation of datasets in Geographic Information System (GIS) format, including conversion of existing data into GIS format and acquisition and collation of new data. In addition to being the basic platform for managing a variety of geospatial data (e.g. information management and data query), GIS has been widely adopted for a wide range of analyses and modelling (e.g. rainfall-landslide correlation and debris mobility modelling) in natural terrain landslide studies.

Other novel applications of digital technology include the generation of ortho-images using digital photogrammetry, boulder identification using image analysis techniques, the use of Interferometric Synthetic Aperture Radar (InSAR) for ground movement detection, and the application of Light Detection and Ranging (LiDAR) for surveying slope related features and for the development of high resolution digital terrain models (Wong 2004a, b). The GEO has also developed a mobile mapping system with GIS capability to facilitate carrying out geotechnical field work (Ng et al. 2004).

This paper presents an overview of development of geoinformatics in the GEO and describes the key datasets that are frequently used in Hong Kong. Notable applications of digital technologies to geotechnical work are also presented, with observations on development and application trends highlighted.
OVERVIEW OF DEVELOPMENT OF GEOINFORMATICS IN HONG KONG

With a land area of about 1,100 km² and a population of about 7 million, Hong Kong is one of the most densely populated cities in the world. As over 60% of the land comprises hilly terrain, a substantial portion of Hong Kong’s dense urban development is located on or near steep hillside (Figure 1). This, together with torrential summer rainfall and sub-tropically weathered ground conditions, poses significant landslide hazards and other geotechnical constraints to developments in Hong Kong.

Following a number of disastrous landslides which resulted in some 150 fatalities in the 1970s, the Geotechnical Control Office (renamed Geotechnical Engineering Office, GEO, in 1991) was set up in 1977 to regulate the whole process of investigation, design and construction of geotechnical works in Hong Kong. As part of the work, the GEO has developed a comprehensive Slope Safety System (Chan 2000), which includes formulating slope safety standards and regulatory framework, checking new geotechnical works, upgrading old substandard slopes, ensuring that slopes are regularly maintained, operating a Landslip Warning System, promoting public awareness in slope safety through public education and information services, and enhancing the appearance and aesthetics of engineered slopes. Implementation of the Slope Safety System also involves partnering with researchers, practitioners and members of the public (Chan et al. 2007).

As a key geotechnical organization and manager of the Slope Safety System in Hong Kong, the GEO has been taking advantages of advances in information and related digital technologies over the years. In particular, the GEO plays a unique role in the following three areas relating to geoinformatics:

1. Compilation and dissemination of geoinformatics datasets – the majority of the key territory-wide geotechnical datasets, such as geological maps, slope catalogue, landslide data, and digital elevation model, are developed and managed by the GEO. The datasets are made available to practitioners and members of the public.

2. Application of geoinformatics – the GEO is a key user of geoinformatics and their applications, which often involve use of GIS, remote sensing and engineering analytical techniques.

3. Development and promulgation of application of geoinformatics – this is achieved through improving the accessibility of the datasets, development and promulgation of customized applications, partnering with researchers and facilitating practitioners in use of geoinformatics.
KEY GEOINFORMATICS DATASETS IN HONG KONG

Introduction

Many geoinformatics datasets have been developed in Hong Kong, and most commonly used datasets are available in GIS format. Some datasets were converted into GIS format by digitizing the available records and acquiring the spatial attributes. Some GIS datasets were collated from field mapping, interpretation of aerial photographs, digital photogrammetry, and other remote sensing techniques. Other GIS datasets were derived from the available data through analysis and modeling. Key geoinformatics datasets that are important to geotechnical and related applications in Hong Kong are described below.

Base Maps

Hong Kong has a comprehensive collection of detailed territory-wide base maps in 1:1,000, 1:2,500, 1:5,000 and 1:20,000 scale. These base maps, available in both paper copy and different digital formats (ArcGIS, dgn, dwg/dxf and TIFF formats), are produced and updated regularly by the Land Information Centre (LIC) of the Lands Department, HKSARG. The topographical data contained in the LIC maps were collated from photogrammetry and topographical surveys. Besides digital topographical maps, the digital LIC maps include other information classes, including land boundary data, geo-reference database, geo-community database and metadata file.

Digital Elevation Model

Digital elevation model (DEM) is essential to 3-D GIS and virtual reality application. In built-up areas where relatively few elevation data (i.e. contour lines and spot heights) are available, where a more accurate DEM is required to cater for local variations of the ground profile, or where the DEM of the pre-development site conditions is required, undertaking site-specific digital photogrammetric analysis and additional topographical surveys can provide supplementary information to enhance the DEM. Where the terrain is covered by thick vegetation, the accuracy of use of photogrammetry to survey the ground profile decreases and this also affects the accuracy of the DEM produced from the LIC map data.

The GEO has constructed two sets of territory-wide DEM. The 5-m grid DEM was built in 2001 using 1:5,000 LIC map data and ground control points (e.g. ground investigation stations). The higher resolution 2-m grid DEM was constructed in 2004 using 1:1,000 scale LIC maps with addition of new algorithm to eliminate flat polygons and dead areas. Improvement has also been made in the provision for break lines, such as drainage lines, roads and slopes, photogrammetric surveys and ground control points to remove artifacts and enhance the accuracy of the DEM in modeling ground features (Figure 2). Related GIS datasets derived from the DEM include shaded relief map and slope gradient maps.

Figure 2: (a) 5-m grid digital elevation model (DEM); (b) 2-m grid DEM with enhancement (Note: roads and man-made slopes are correctly modelled)
Ortho-rectified Images
Hong Kong has a comprehensive collection of aerial photographs, taken at about 1,200 m to 3,600 m high, dating back to as early as 1918. In the early years, a set of aerial photographs covering Hong Kong was taken about once every five to ten years. Since the early 1970s, the frequency has been increased to about once a year.

By application of digital photogrammetric techniques, conventional aerial photographs can be converted into ortho-rectified images. Such images are true to scale and position-accurate, which can supplement survey plans (Figure 3). Ortho-rectified images contain rich visual and spectral details of the ground at the time of taking the aerial photographs, and are most suited for use in field reconnaissance, mapping and remote-sensing analysis. The ortho-rectified images also serve as an important geometrically corrected and spatial enabled datasets for detecting temporal changes of landform, ground and man-made features in Hong Kong, where rapid urban development has been taking place. As ortho-rectified images are in digital and geo-referenced format, they can be integrated into GIS environment together with other spatial data for a range of novel GIS and remote-sensing applications as described in later sections.

Figure 3: Comparison of (a) ordinary aerial photograph and (b) ortho-rectified image

Catalogue of Man-made Slopes
To collect comprehensive information on all sizable man-made slopes in Hong Kong, the GEO started a project ‘Systematic Identification and Registration of Slopes in the Territory (SIRST)’ in July 1994. Under this project, sizable man-made slopes and retaining walls (collectively referred to as ‘slopes’) were systematically identified, catalogued and registered. For slopes without documentary records, aerial photographs and topographical plans were used to identify slopes for registration. The Catalogue of Slopes is regularly maintained and updated, and it now contains about 57,000 man-made slopes (GEO 2006). Technical details of the compilation of the catalogue are described in Lam et al. (1998).

Apart from the physical information on slopes, information on the maintenance responsibility of individual slopes is also essential to arrangement of maintenance and repair works. The Lands Department with assistance from the GEO has carried out a project ‘Systematic Identification of Maintenance Responsibility of Slopes in the Territory’ (SIMAR) to identify the ownership and maintenance responsibility for all the slopes in the Catalogue of Slopes.

The relevant datasets, including the GIS polygons of the 57,000 man-made slopes and the corresponding slope data (Figure 4), form an important inventory of information on man-made slopes in Hong Kong. The GEO manages and disseminates the data through different channels that facilitates their use by the general public and geotechnical practitioners.
Reported Landslide Incidents
On average, about 300 landslides are reported to the GEO every year. These reported landslide incidents, which are inspected and recorded by the GEO, are mostly landslides on man-made slopes within or bordering developed areas. The work has been carried out systematically since 1984, and ad-hoc records of individual landslides on man-made slopes in the earlier years were also available. These landslide records have been digitized and converted into GIS format (Figure 5). The dataset provides information on the performance of man-made slopes and on the location, time, scale, dimensions and consequences of landslides affecting developed areas in Hong Kong over the past 20 years.

Natural Terrain Landslide Inventory
The Natural Terrain Landslide Inventory (NTLI) was compiled by the GEO from interpretation of aerial photographs taken at about 2,400 m or above. The location of each identified natural terrain landslide crown was recorded and the centre-line of the debris trail
was marked with a line (Figure 6). Up to the year 2003, the NTLI has catalogued some 30,000 landslides, about 11,000 of which are considered recent (i.e. those occurred within the period of aerial photograph coverage, typically after the 1950s). The NTLI has been an important source of information for studies on natural terrain landslide hazards in Hong Kong.

Between 2004 and 2007, the GEO undertook a comprehensive aerial photograph interpretation using both high- and low-flight aerial photographs (at <2,400 m or below) taken between 1924 and 2003, with improved resolution and temporal coverage. The new inventory, referred to as the Enhanced Natural Terrain Landslide Inventory (ENTLI), contains information on about 105,000 landslides (about 14% are recent) occurred on natural terrain. Information recorded for each landslide record includes the dates of the aerial photographs when the landslide was first observed, width and length of the landslide scar, slope gradient, and nature of vegetation cover across the landslide source area. The relict landslides are further classified according to the degree of certainty of interpretation, based on specific terrain characteristics.

![Figure 6: Natural terrain landslide inventory (ortho-rectified image and LIC map as back-drop)](image)

**Geological Maps**
A territory-wide detailed geological survey mapping, carried out by the GEO, commenced in 1982. Geological maps covering all of Hong Kong have been completed at 1:100,000 and 1:20,000 scale, and maps in 1:5,000 scale are available for selected areas with complex geological conditions. The maps and the accompanying geological memoirs form an authoritative spatial dataset on the geology of Hong Kong. Rock lithologies and superficial deposits in both land and offshore areas are compiled in GIS layers. Each lithology or rock type is stored with symbology and linked to the digital base map for reference (Figure 7).

A regional gravity survey of Hong Kong was commissioned in 1991, to identify deep geological structures beneath Hong Kong. It is now thought that the regional gravity gradient that crosses Hong Kong is the result of a major northeast-trending structural discontinuity in the earth's crust at a depth of at least six kilometres. A regional magnetic survey of the
offshore area of Hong Kong was completed in 2000 and the results were published at 1:100,000 scale as part of the Memoir on the Pre-Quaternary Geology of Hong Kong (Sewell et al. 2000). The data are particularly useful for identifying the locations of faults in offshore areas, several of which are continuations of structures previously mapped during onshore surveys. Shallow marine seismic data involving 5,000 km of boomer seismic survey records allowed a detailed stratigraphy of the offshore superficial deposits to be constructed (Fyfe et al. 2000). These data form part of the geospatial datasets.

Figure 7: Geological information with other GIS data

**Terrain Landslide Susceptibility Classification**

The first territory-wide landslide susceptibility map was prepared by the GEO in 1998, based on correlation of landslide susceptibility with slope angle and geology (Evans & King 1998). The natural terrain landslide data were taken from the NTLI up to the year 1994. The analysis used a DEM created from the 1:20,000-scale topographical plans converted first into a Triangulated Irregular Network (TIN) model and then into a grid model. The period of landsliding evident in the aerial photographic records was about 100 years. This allows a straightforward conversion from landslide density to landslide frequency. The calculated landslide densities were increased by 20% to allow for landslides not recognized in the NTLI. Five susceptibility classes were defined: very low, low, moderate, high and very high with densities varying from <10 to >100 landslides per km² corresponding to frequencies varying from 0.1 to >1 landslide/yr/km² (Figure 8).

Figure 8: Terrain susceptibility classification
Rainfall Data
In collaboration with the Hong Kong Observatory (HKO), the GEO has been operating an extensive network of automatic rain gauges that record real-time rainfall data and form part of Hong Kong’s Landslip Warning System since 1984. The GEO network comprises 86 rain gauges located throughout Hong Kong, and is supplemented by another 24 rain gauges operated by the HKO. The rain gauges are equipped with data-logging facilities and are powered by solar energy. The rainfall data are transmitted wirelessly at 5-minute interval to the GEO, where the data are downloaded, analyzed and disseminated (Figure 9).

The historical rainfall data available since 1984 have been processed by GIS geostatistical analysis and converted into 5-minute rainfall data covering the whole of Hong Kong in GIS raster cells. The spatial distributions of maximum rainfall at different durations have also been derived for major historical (since 1984) rainstorms in Hong Kong and these are managed as GIS datasets by the GEO (Figure 10).

![Figure 9: Display of real-time rainfall data](image)

![Figure 10: Derived spatial distribution of a rainstorm (24-hr rainfall on 26.9.1993)](image)

Squatter Structures and Related Dataset
Squatter structures are a specific class of facilities vulnerable to landslide risk, and they require particular attention for effective risk management. Since the early 1980s, the GEO has
been undertaking systematic geotechnical inspections to identify squatter structures that warrant priority clearance due to landslide hazards. A GIS inventory of squatter structures and their related attributes has been compiled by the GEO to facilitate their landslide risk management (Figure 11). The dataset also contains information on the land status of the structures and on locations where safety clearance has previously been recommended.

**Figure 11: Squatter structures**

**Other Datasets**
A range of other datasets, which are typically managed by other Government departments, are available in Hong Kong. These datasets are not geotechnical in nature but are often adopted, and hence form an important source of information, in geotechnical application. Examples include the 1:75,000-scale land-use and vegetation classification map, land-use zoning, census data, building height, boundaries of District Councils, location and reference numbering of lamp posts, and underground utilities.

**MANAGEMENT AND DISSEMINATION OF GEOINFORMATICS**

**Modes of Arrangement in Hong Kong**
A community with diverse interest and needs, as in the case of Hong Kong, comprises different types of user of geoinformatics. For example, the general public and geotechnical professionals tend to have different information requirements and different levels of technical capability in accessing and applying geoinformatics. While there are three different modes of management and dissemination of geoinformatics in the GEO, the information that is being served could come from the same source.

**Slope Information System**
The GEO operates the Slope Information System (SIS) for service of the Catalogue of Slopes and other slope-related information to the public, as well as to professional users. Pertinent information on all registered man-made slopes is linked by the slope registration number to a textural database with site photographs showing the slope conditions and configuration. Other relevant information, such as slope location, physical dimensions, drainage conditions, history of development and maintenance records, are stored in the Oracle database. The SIS also serves as the in-house platform for managing project-related information.
The GIS graphical search interface including the web-service functionality is powered by GeoMedia. Besides serving the information via a Local Area Network through the Government’s Intranet system, the SIS also serves data through the Internet via the Hong Kong Slope Safety Website (http://hkss.cedd.gov.hk), where members of the public have free access to making on-line spatial queries and browsing the slope information (Figure 12). The SIS itself has limited GIS functionalities for advanced GIS analysis and modeling.

Launching of the SIS in 1999 marked a milestone in the application of GIS, which succeeded in integrating a wide range of spatial and textual datasets, and efficiently disseminating vast amount of slope-related information to practitioners and the general public via the Internet (Mak et al. 2001). In 2003, the SIS received the ‘Geospatial Achievement Award – Certificate of Merit’ by the Intergraph Corporation, in recognition of the extraordinary contribution of the SIS to the public through innovative application and implementation of geospatial technology for free dissemination of slope information on the Internet.

**Geological Modeling System**
The GEO also operates a high-end GIS, namely the Geological Modeling System (GMS), for professional application. The system was set up in the early 1990s as a GIS platform for management of geotechnical data and geological modeling. The GIS datasets that are accessible to the system and its GIS functionalities have expanded considerably over the years. The GMS has become a core GIS in the GEO for professional application that covers the full range of information service, analysis and modeling. Although the SIS and GMS interact with users as two separate platforms, their back-end data are from the same sources that are centrally managed by the GEO.

The system architect of the GMS comprises a suite of ESRI software as GIS and graphic engine, and Oracle as relational database. Its hardware includes high-powered servers, workstations and digital photogrammetric facilities. Workstations of the GMS are connected to dedicated GIS servers via different configurations, including Local Area Network, Internet and mobile connection, for enterprise GIS application powered by Spatial Database Engine (ArcSDE) and Internet Map Server (ArcIMS) (Figure 13). Due to security control and copyright requirements for some of the datasets, the GMS only permits restricted access to pre-registered or nominated users.
Tailor-made Systems
From time to time, tailor-made GIS platforms could be developed for specific use of the geoinformatics datasets. Three selected platforms operating in the GEO are described below to illustrate the arrangement.

Aerial Photograph Library Management System (APLMS)
The GEO operates an aerial photograph library, which holds more than 150,000 historical aerial photographs. The library is open to all government staff and consultants working on government projects, and about 6,000 aerial photographs are loan out each month. In the past, search of relevant aerial photographs covering a site was done manually by going through a set of flight plans. As the number of photographs held in the library continues to grow, the manual search becomes difficult and time-consuming.

The APLMS was developed by the GEO to enable spatial search of the relevant aerial photographs and provide library management functions, which assist users to identify the photographs and their availability (Figure 14). The APLMS runs on an ArcView 3.2 platform that is connected to the data server via a Local Area Network. It has also been incorporated into the Internet module of the GMS, which allows access to the APLMS functionalities using an ordinary web-browser through the Internet. A new version is being developed on an ArcIMS platform.
**Digital Geotechnical Information Unit (DGIU)**

The GEO manages a Geotechnical Information Unit (GIU), which contains a comprehensive collection of data and reports from ground investigations (GI) throughout Hong Kong. The ground investigation data and reports were provided by all public works project and selected private development projects over the past 20 years, for shared use among professionals, researchers and members of the public. Up to 2006, the GIU holds a total of about 56,000 reports (covering GI and laboratory testing), of which about 24,000 are GI fieldwork reports containing records of about 300,000 ground investigation stations. Recent records of about 40,000 are available in digital Association of Geotechnical and Geoenvironmental Specialists Data Interchange Format (AGS format).

A system called the Digital Geotechnical Information Unit (DGIU) is being developed by the GEO on an ArcIMS platform. With the system, existing GI information held in GIS servers via Local Area Network can be located and the relevant digitized GI records can be viewed on computers (Figure 15). The DGIU will be launched for trial use in December 2007. Digitization of the remaining reports will be completed in the late 2008 when the DGIU will be in full operation.

![Figure 15: Digital geotechnical information unit: (a) user interface; (b) GI report](image)

**Planning Comment Geographic Information System (PCGIS)**

The system stores and manages GEO’s geotechnical input in checking land-use planning and development proposals. The layout of the proposed planning and development sites are recorded as a GIS layer, and the relevant documents of GEO’s geotechnical input are appended as digital files hot-linked to the GIS layer (Figure 16). The PCGIS allows fast retrieval, based on either spatial or textual queries, of previous cases where geotechnical input has been provided. It saves time and minimizes human error in information search. The system operates on a GIS platform developed by the GEO using MapObject. It accesses data held in the GIS servers via Local Area Network, and it also serves as a relatively ‘light weight’ GIS platform for searching, browsing and editing the GIS datasets available in the GEO.
APPLICATION OF GEOINFORMATICS AND RELATED TECHNOLOGIES

Development of Capability
By nature, virtually all geotechnical data contain spatial attributes on their geographical location (x, y, and z) and on the geometry of the ground/object (e.g. point, line or polygon) represented by the data. Managing the data in GIS would register the spatial attributes and permit the use of the attributes in GIS-related application, resulting in improved capability and efficiency.

Since the early 1990s, the GEO has been a key party in developing geotechnical spatial datasets and GIS capability, and in promoting GIS application to geotechnical work in Hong Kong. In the early years, the development and application work focused on the following three areas:

1. Compilation and management of GIS datasets, including conversion of existing data into GIS format and collation of new data, and maintenance and dissemination of the data.

2. Development of GIS systems and capability, such as (i) setting up software and hardware systems, (ii) developing GIS skills among professional and technical users, (iii) integrating GIS datasets and setting up centralized database for enhanced system management, and (iv) acquiring Intranet, Internet and mobile GIS capabilities for ‘enterprise-based’ and other functional GIS applications.

3. GIS applications, which mainly focused on data management and information services. A number of state-of-the-art systems were set up in the GEO over the last 10 years, e.g. SIS, GMS, APLMS, DGIU and PCGIS.
GIS Related Applications
While GIS data management and information service remain important, the trend in recent years is to adopt more advanced GIS functionality to deal with geotechnical applications in Hong Kong. This is partly the result of gradually building up of GIS capability among the geotechnical profession, and more importantly the increased demand and recognition for use of GIS in solving geotechnical problems.

With advances in information technology, the boundaries between different systems have progressively been removed. Most datasets that were created for individual systems are now integrated, centrally managed and made available for use in different platforms and applications. The capability of hardware and software systems has also been greatly enhanced with advances in digital technology. Particular attention is given to data compatibility and to development of enterprise-based, scaleable tools and applications. As a result, the systems become individual platforms where specific applications are undertaken. Some notable applications are described in the following sections.

Advanced GIS Query and Publication
This commonly includes in a geotechnical desk study to examine the available geotechnical data, review the site history and assimilate the key information for presentation. For example, recent landslides and new developments can be identified from reviewing aerial photographs of different vintages (Figure 17). Advanced GIS search would enable users to query and retrieve data that meet certain prescribed criteria or geographical relationship. Examples of such application include delineation of area of deep rock weathering using ground investigation data, search of man-made slope features for stability assessment, and identification of sites affected by historical natural terrain landslides. Map data can be published using pmf format on ArcReader platform.

![Figure 17: Use of GIS in geotechnical desk study: (a) Site in 1963; (b) Site in 2000](image)

GIS Analysis
GIS analysis can be performed efficiently to examine the relationship and correlation among different spatial data, which are difficult to analyze using conventional means. It is now commonly used in natural terrain landslide susceptibility analysis, as part of natural terrain hazard study (e.g. Evans and King 1998; Dai and Lee 2002; Halcrow 2003; OAP 2003; Wong 2004). GIS analysis offers a unique capability in geotechnical research and development work involving spatial analysis of geotechnical data. Figure 18 shows an example where the correlation between natural terrain landslide density and rainfall intensity in Hong Kong has been established using GIS analysis together with GIS-based geostatistics (Ko 2003).
GIS Modeling

Performing GIS-based geotechnical analysis and numerical modeling based on application of engineering principles and governing physical laws has become increasingly important. Such application integrates engineering analysis with GIS, and is a powerful modeling tool, particularly for dealing with geotechnical subjects that involve the analysis of the geographical and engineering attributes of a large amount of spatial data. Examples of such application in modeling the mobility of landslide debris (Kwan et al. 2007) and natural terrain landslide quantitative risk assessment (Wong 2005) are shown in Figures 19 and 20, respectively. In the latter, hazard and consequence models are incorporated into the GIS calculation of risk using the relevant spatial data of the catchments and facilities at risk.
Rainfall – Landslide Warning System
The GEO and HKO operate a Landslip Warning System since 1978, to alert the general public to reduce their exposure to possible danger from landslides, and to trigger the operation of an emergency system within government departments that mobilizes staff and resources to deal with landslide incidents. A simplified GIS approach was used to obtain territory-wide correlation between slope failure rate, maximum rolling 24-hour rainfall and slope characteristics for soil cut slopes (Yu et al. 2004). This new approach has been adopted since 2004 for predicting the number of landslides. Landslip Warning will be issued when the landslip warning level (i.e. when the estimated number of landslide is 15 or more) is expected to be reached.

Mobile Location-based Application
Fieldwork forms an important component of geotechnical practice. GIS can now be brought to site and applied to fieldwork, by uploading the relevant datasets onto a mobile GIS platform that operates on a pocket computer. When integrated with a Global Positioning System (GPS), a mobile GIS system can guide on-site navigation to the point of interest and can record the location of features identified on site. In addition, the spatial data relevant to the site can be retrieved for location-based applications.

The GEO has pioneered the development of a state-of-the-art GIS-GPS mobile mapping system that also incorporates the use of ortho-images (Ng et al. 2004). The system is equipped with wireless telecommunication via the Internet with GEO’s GIS Internet Map Server, for GIS data transfer for use in geotechnical fieldwork (Figure 21). This system was granted an international ‘Special Achievement in GIS Award’ by the ESRI in 2002 in recognition of its technological advances and benefits to the engineering field.

![Figure 21: Mobile mapping system: (a) DGPS equipment; (b) system in action](image)

3-D Visualization and Virtual Reality Applications
To accompany the DEM, virtual reality (VR) models have been generated by combining with ortho-rectified aerial photographic images. 3-D ground model can now be generated on desktop computers. These VR models are useful in visualization of the landform and urban development (Figure 22), and identification of geotechnical features such as past landslides, boulders, tension cracks and man-made slope features (Figure 23). This capability enhances the quality of geotechnical studies, e.g. in reviewing historical landslides and terrain evaluation (Wong 2004b). VR animation and computer ‘fly-through’ can also be produced for presentation and evaluation purposes.

634
Remote Sensing Related Application
The latest development in application of geoinformatics includes integration with remote sensing technologies, such as digital photogrammetry, image processing, Light Detection and Ranging (LiDAR), and Interferometric Synthetic Aperture Radar (InSAR).

Image Processing
Image processing techniques have been adopted in parallel with digital photogrammetry for feature identification, extraction and change detection (e.g. Jensen 1996). An example of such application to mapping of boulders in Hong Kong is given in Figure 24. In collaboration with the Hong Kong Polytechnic University, a software system has been developed for such purpose. Ng et al. (2003) show that by integrating with digital photogrammetry and human-machine interaction, image analysis data can be used to map boulder distribution and extract boulder properties (e.g. size and shape), which are the key parameters for assessing boulder fall hazard.
Laser Scanning
Land-based (or Terrestrial) Light Detection and Ranging (LiDAR) technique, commonly denoted as ‘Laser Scanning’ is increasingly used for landslide mapping (e.g. Jones 2006) and rock slope studies (e.g. Rosser et al. 2005). The Laser Scanner operated by the Survey Division of CEDD has the capability of measuring 3-D point clouds of objects within about 150 m along the line-of-sight. The laser scanner emits thousands of laser beam pulses per second for measuring a ‘window’ of 3-dimensional surfaces. The positional accuracy is within 6 mm in a 50-m range. The point clouds, apart from providing spatial information on their x, y, and z coordinates, contain an intensity signal of the laser reflection and hence present a 3-D false-color digital model of the scanned object. Some novel geotechnical applications of laser scanning include:

(a) Construction of high-resolution DEM – given the high sampling density, DEM produced by laser scanning can enhance the quality and supplement the DEM produced from the available topographical maps and from digital photogrammetry. This method is particularly useful where physical access to the survey site is difficult or dangerous, e.g. a new landslide scar (Figure 25).

(b) Compilation of 3-D digital models of slopes, debris-resisting barriers, structures and other geotechnical features (Figure 26) – this assists construction monitoring and provides an accurate and detailed VR records for use in future maintenance and modification work. Similar application of laser scanner in surveying a full-scale test of fill slope failure has been reported by Kwong (2003).

(c) Movement monitoring of slopes and structures – movement can be detected and monitored by comparing laser scanning results obtained at different times, with some common observation points to ensure accuracy and efficiency.

(d) Rock slope mapping and rock joint survey – by judiciously matching and analyzing the laser scanning point clouds.
Airborne LiDAR

Airborne LiDAR, which can survey a large area efficiently and at competitive cost, is subject to active technological development. An important recent development is the use of a multi-return LiDAR to measure multiple returns for each laser pulse that can cover circa one metre in diameter on ground (Figure 27). With the use of an advanced numerical algorithm, the last returns that come from the ground surface are extracted by filtering out other returns from vegetation and building structures (a technique known as ‘virtual deforestation’). Hence, the system has the capability of mapping the ground surface of vegetated terrain (e.g. McKean & Roering 2004; Schulz 2007). The technique has been used to produce fine-scale topographical maps and DEM typically with grid size of about 1 m.

The GEO conducted a pilot airborne LiDAR survey in December 2006 for Hong Kong Island, to assess the ‘virtual deforestation’ capability on heavily vegetated hillsides and the ability to detect slope features behind tall buildings. The technical requirements of the survey were sampling interval at 1.3 m and horizontal and vertical data accuracies at 0.3 m and 0.13 m respectively. An aerial survey and mapping service provider from Australia was awarded the survey contract which included deploying multi-return LiDAR equipment for the survey and processing of the data acquired.

Data verification and evaluation of the pilot survey results shows that the accuracy of hard surface was generally about ±5 cm whereas the accuracy of vegetated surface was generally
about ±10 cm. In areas of dense shrubland, where laser penetration was relatively low, the accuracy could be lower. This high resolution ground data allows clear definition of ground features for terrain evaluation and the interpretation of landslide morphology (Figure 28). The pilot survey results are also being used to plan future surveys for the remaining parts of Hong Kong. It is anticipated that airborne LiDAR will become more widely used in Hong Kong and will bring about enhanced remote sensing capability that will facilitate geotechnical assessment and design, including studies of natural terrain hazards.

**Figure 27:** “Virtual deforestation” capability of airborne LiDAR survey

![Airborne LiDAR Survey](image)

**Figure 28:** (a) Ortho-image with landslide locations; (b) LiDAR DEM showing detailed morphology of landslide scars

**Interferometric Synthetic Aperture Radar (InSAR)**

InSAR is an emerging remote sensing technology that could measure ground displacements with millimeter-level accuracy. This technology has been successfully applied to measurement of surface movement over large area induced by earthquakes (e.g. Massonnet et al. 1993), volcanic activities (e.g. Lu et al. 2002) and ground subsidence (e.g. Fielding et al. 1998). Some cases of application to detection of slope movement have been reported (e.g. Strozzi et al. 2001). InSAR is potentially a low cost, high accuracy remote sensing technique for geotechnical use, particularly for ground and slope movement detection.
InSAR has been tried with encouraging results in Hong Kong in detection of settlement (Ding et al. 2004). In collaboration with the Hong Kong Polytechnic University, the GEO has recently completed a trial application of InSAR for detection of slope movement and ground deformation at three selected sites in Hong Kong. The trial study shows that use of the available space-borne SAR images in InSAR analysis for the typical urban setting in Hong Kong is subject to some constraints (Wong 2004b) as given below.

(a) Many of the available SAR images have large Doppler Centroid Frequency and low coherence, particularly for long time-span. These result in poor quality interferograms, which affect the reliability of InSAR analysis.

(b) SAR images have low spatial resolution (each pixel typically >10m). This limitation is particularly relevant to small ground features (e.g. a relatively small-sized man-made slope) or areas where local changes in ground profile and ground movement are significant.

(c) In Hong Kong, geometric distortion can be a serious problem due to the steep terrain. The relatively humid environment and presence of thick vegetation can result in significant atmospheric and temporal decorrelation effects.

The quality and availability of SAR images would improve as satellites with high precision onboard positioning computer and stable orbits are launched. Development of airborne and land-based InSAR would reduce noise effects and enhance the accuracy and spatial resolution of InSAR results. InSAR technologies, e.g. use of corner reflectors, filters and permanent scatter techniques, are evolving. Nine corner reflectors installed at the trial sites in Hong Kong provide an opportunity for testing the improvement that may be achieved and for developing the long-term potential for geotechnical application of InSAR in Hong Kong.

CONCLUSIONS
The GEO has a comprehensive collection of geoinformatics datasets, and different arrangements are in place for management and dissemination of these data. The geoinformatics datasets are being applied, in combination with advances made in related digital technologies, to a diverse range of geotechnical applications. As a result of integration of datasets, hardware and software, there is a shift of emphasis from development of systems to development of applications in recent years. The trend of application has evolved over the years from conventional information services involving searching and browsing to advanced spatial analysis, modeling and remote sensing assessment.

At the GEO, the development work and the associated application of novel digital technologies have led to improved capability and efficiency in carrying out geotechnical work, and have helped revolutionize the way in which landslide risk management is conducted. Keeping awareness of the development of emerging technologies and exploring customization of these technologies would be beneficial to conducting geotechnical work effectively and efficiently. The scope of application will continue to expand as digital and information technology further improves and becomes more accessible to professional users and members of the public.
REFERENCES
Chan, R. K. S., Mak, S. H., and Au-Yeung, Y. S. (2007). “Partnering with the community to reduce landslide risk in Hong Kong over the past thirty years.” Proc., Geotechnical Advancements in Hong Kong since 1970s, Hong Kong Institution of Engineers, Geotechnical Division, 183-195.


ACKNOWLEDGEMENTS

The paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, Government of the Hong Kong Special Administrative Region.
PREDICTION OF RAINFALL-INDUCED LANDSLIDES IN UNSATURATED GRANULAR SOILS FOR SETTING UP OF EARLY WARNING SYSTEMS

Luciano Picarelli
Analisi e Monitoraggio dei Rischi Ambientali (A.M.R.A. s.c.a.r.l.), Napoli

Pasquale Versace
Dipartimento di Difesa del Suolo, Università della Calabria

Lucio Olivares and Emilia Damiano
Centro Interdipartimentale di Ricerca in Ingegneria Ambientale (C.I.R.I.AM.)
Dipartimento di Ingegneria Civile, Seconda Università degli studi di Napoli

Abstract: Early warning is being used more and more for protection against natural risks. In some cases, as for extreme meteorological events, it has been proven to be efficient for risk mitigation; in other cases, as for rapid landslides, the use of early warning is conditioned by the very short lead time which is available between the event and its impact on exposed elements. In particular, in the case of rainfall-induced landslides, early warning should rely on monitoring of rainfall and on empirical relationships between this and landslide occurrence, forcing to launch the warning signal before the event when the probability of occurrence is considered to be high enough. Because of the complexity of the problem and of the risk of false or missing alarms, landslide prediction should bear on a clear knowledge of the mechanics of slope failure, with the support of advanced technologies for monitoring and data transmission and of reliable procedures for a timely analysis of collected data. This paper examines some of these problems, with particular reference to prediction of landslides, reporting experience on events involving pyroclastic soils.

INTRODUCTION
The risk of landslide is extremely variable. In fact, slope movements present a wide range of velocity, size and run-out, thus their magnitude and impact on exposed elements can be either very low or very high, depending on site conditions, geomaterial involved and other factors. Landslide velocity is presently considered to be a key factor for risk assessment. It can range between some tens of metres per second (as for rock falls and avalanches, debris flows and flowslides) and some millimetres per year (as for active slides in clay and for some lateral spreads): landslide velocity not only affects its destructiveness, but also the procedures to adopt for risk mitigation. For instance, the measures that are usually adopted for stabilization of slow to moderate landslides in clay, cannot be applied to rapid landslides due to the very short time which is available for any type of intervention. Therefore, in some cases, the safety of individuals depends only on luck, unless preventive passive works have been built in advance, or procedures for a timely people evacuation have been adopted.

THE CONCEPT OF EARLY WARNING
Early warning can be defined as the whole of the actions to be taken during the lead time of whatever catastrophic event. Gasparini et al. (2207) define the lead time as the time interval
elapsed between the moment of precursor occurrence, when the event is reasonably certain
and the moment of its actual occurrence. In more general terms, early warning is the
 provision of timely and effective information allowing individuals exposed to a hazard to
take action in order to avoid or reduce the risk. In order to do it, rapid disaster information
systems are of crucial importance.

In previous years, early warning systems have been employed for protection against some
natural risks. In some cases, as for heavy meteorological events (tornadoes, typhoons, etc.),
for volcanic eruptions and for tsunami, they prove quite efficient. In fact, in such cases, the
length of time comprised between the initiation of the event and the impact on exposed
elements (as in heavy meteorological phenomena or tsunami), or between the detection of
first reliable precursors and initiation of the event (as in volcanic eruptions or floods
involving large basins), is long enough to take action as evacuation or protection of some
crucial structures and infrastructures, depending on the event. In other cases (as for
earthquakes, floods involving small basins, rapid landslides), the time between the event and
its impact is extremely short thus adoption of early warning procedures may require special
solutions. As a matter of fact, in these cases, early warning is adopted only for limited goals
or the signal is launched well before the event, when its probability is high enough but the
event is not really certain. This last approach implies a high subjectivity in the decision and
can determine false or even missing alarms, posing additional problems concerning the social,
economic and legal consequences of mistakes.

For earthquakes, the occurrence of the event is unequivocal, as the lead time must be
evaluated from the time when the first seismic waves released by the source are detected.
Since strong ground shaking is provoked by shear and surface waves whose travel speed is
about half the speed of primary waves, and these are much slower than electromagnetic
signals transmitted from the epicentre by cable or wireless, transmission of information
regarding the faster primary waves and a first real-time analysis of the effects of these may
provide warnings concerning the approaching ground shaking (Gasparini et al. 2007). As a
matter of fact, the real-time analysis of signals can be exploited to locate the epicentre, to
assess the magnitude of the quake and to estimate the distribution of ground motion. This
analysis requires some little time, thus the remaining length of time to take action depends on
the distance of the area to be protected from the epicentre. If such a length of time is at least a
few seconds to tens of seconds, automated actions can be taken, as turning off of electrical
power, shutdown of pipelines and gas lines, arrest of trains, activation of special pr
otection systems of critical structures, as nuclear power plants, etc. This is the case of several large
towns, as Naples, which are situated tens of kilometres from the most likely epicentral areas.

Early warning systems could also be used to mitigate the risk posed by rapid landslides. Since
the time elapsing between the onset of slope failure and its impact on exposed elements is
typically in the order of tens of seconds, the problem is similar to the one posed by
earthquakes, but even more complicated in areas lacking any type of instrument able to
recognize the occurrence of the event because the lead time is equally short, and the landslide
can occur everywhere. Research in this field is active, even though it is just the beginning.
Usually, the relationship between landslide and rainfall intensity or duration is investigated
and empirical thresholds are provided. In particular, these correspond to crucial moments
such as: precursor forecasting, precursor occurrence, event initiation, impact on people and
elements. The precursor is generally rainfall, but any other strictly relevant and measurable
factor providing indication about the probability of occurrence of the landslide can be
adopted.
This paper focuses on the methods which can be used in the case of rainfall-induced landslides in granular soils, with particular reference to debris flows occurring in pyroclastic materials outcropping in Campania Region, which represent one of the most severe natural hazards in Italy and provoke a large number of victims and extensive damage (Picarelli et al. 2007). It shortly reviews the mechanisms of slope failure and post-failure with particular reference to the indicators of failure, then discusses the lines along which new and more refined procedures can be developed for timely prediction of slope failure.

INDICATORS OF IMPENDING RAINFALL-INDUCED SLOPE FAILURE IN UNSATURATED GRANULAR SOILS

Data from Experience and Experiments
In general, it is thought that slope failure in granular soils is preceded by too small and fast displacements to be considered as indicators of impending failure. However, this point deserves some insight, because of the relevance of the topic. Experiments on full-scale or even small-scale physical models can be extremely useful in this respect. This is one of the goals of an experimental program which is being carried out at the Geotechnical Laboratory of C.I.R.I.A.M. with particular reference to rainfall-induced landslides (Figure 1).

Figure 1: The flume available at the geotechnical laboratory of C.I.R.I.A.M.

Figure 2 reports the main results of two tests performed on a 10 cm thick, 1.2 m long, 40° model slope made up with very loose (e=2.70, on the left side) or relatively dense (e=1.85, right side) unsaturated volcanic ash, resting on a rough impervious base. The soil is a cohesionless silty sand (the Cervinara ash) with a friction angle of 38°. In the experiments, the average initial suction, $u_a$, was respectively 60 kPa and 50 kPa, corresponding to an apparent cohesion of around 9 kPa (Olivares 2001). Failure was provoked by uniform artificial rainfall. The profile of suction recorded with superficial and deep tensiometers (uppermost diagrams) reveals the evolution of the wet front from the ground surface to the base of the layer, which is reached when the deep tensiometers indicate vanishing of suction because of full saturation (Figure 2a). Since then, in the last few minutes before failure, the probes installed at the base of the layers start to measure a positive pore pressure (Figure 2c).
The effects of suction decrease are shown in the intermediate diagrams, which display a settlement of the ground surface, measured with laser transducers. The average volumetric strain of loose soil attains a value of as high as about 8%; in contrast, the dense soil shows a negligible strain, or some dilation just prior to failure. This is confirmed by personal experience. In fact, even when general slope failure is not attained, man made works located on sloping grounds may reveal deformation and fissuring provoked by infiltration, while the ground surface is subjected to cracking (Picarelli et al. 2008a). The lowermost diagrams in Figure 2 show that failure in loose soil is followed by a sudden pore water pressure increase which can attain a peak value very close to the total stress, suggesting the occurrence of liquefaction. The same does not occur in dense soil.

Significant volumetric deformation of soil prior to failure has been recognized also by Take et al. (2004) through small-scale centrifuge tests. Similar data about the behaviour of loose soil have been obtained by Ochiai et al. (2004) and by Moriwaki et al. (2005) through full-scale tests.

Figure 2: Results of a flume test carried out on the Cervinara volcanic ash (loose soil on the left side; dense soil on the right side): (a) Suction measured at two different depths; (b) settlement of the ground surface; (c) pore pressure recorded at the base of the layer (modified from Damiano 2004)
Such data demonstrate that either suction decrease (or the correlated value of water content) or soil deformation, can be used as indicators of impending failure. However, while suction must be correlated to the other factors (such as the slope angle or the friction angle) to provide information about slope stability, soil deformation must be evaluated as such.

The role of suction on slope stability in unsaturated granular soils has been clearly demonstrated by several authors (see, for instance, Lim et al. 1996). Figure 4 shows some data regarding suction measured in the years 2002-2003 and 2006-2007 in a 2.5 m thick, 40° sloping deposit covering fractured limestones (Figure 3). The cover consists of alternating layers of pumice and volcanic ash, the same which has been used for flume tests discussed above. The average void ratio of the volcanic ash is around 2.0. Suction continuously fluctuates as a consequence of changes of humidity and of rainfall; it reaches values as high as 80 kPa, corresponding to an apparent cohesion of about 10 kPa, but can rapidly drop to a few kPa. This strongly affects the stability conditions of the slope. As a matter of fact, on December, 16th, 1999, the slope just adjacent to the instrumented zone failed, as a consequence of rainfall which lasted for two consecutive days, amounting to 329 mm (Olivares & Picarelli 2003). The mechanism of failure was very similar to the one shown in Figure 2 for loose soil (flowslide).

The discussed remarks about phenomena which precede slope failure in unsaturated loose pyroclastic soils, either in model slopes or in natural slopes, stimulated a research aimed at developing methods to capture in advance any indicator of impending slope failure, to exploit in the design of early warning systems. A team composed by experts in geotechnical and in hydraulic engineering, as well as in optoelectronics, is closely working at C.I.R.I.AM. on such a project (Olivares et al. 2008; Damiano et al. 2008). Following this idea, flume tests are being carried out on small-scale slopes instrumented with either “usual” laboratory sensors, such as micro-tensiometers, probes for pore pressure measurement, laser transducers for displacement readings as well as with video-cameras, either instruments conceived to be used in site for warning purposes, as TDR probes and optical fibres.
Figure 4: Rainfall, h (uppermost part of the diagrams), and suction, s (lowermost part), measured at different depths in the slope of Figure 3: (a) 2002-2003; (b) 2006-2007

Capturing of Failure Indicators

The use of indicators for prediction of landslides is an experimented procedure (Saito 1965; Voight 1998). As an example, in recent years, remote sensing techniques to capture indicators of landslides have been developed and strongly improved (Antonello et al. 2004). With the aim to extend the use of such techniques to civil protection, the European Community has launched a number of preparatory activities in the framework of GMES (Global Monitoring for Environment and Security) for the development of an Emergency Response. Among the other products, it is worth to mention the so called Rapid Landslide Mapping (RSLM) which will be carried out on a time scale spanning a few days, for mapping of the spatial distribution of unstable areas and supporting the decision-making activities of civil protection authorities. Such a technique is based on the integration of multi-interferogram InSAR analyses of historical SAR scenes needed to detect ground displacements before catastrophic events, with conventional DInSAR processing of the last SAR acquisitions covering the destructive event. The analysis of ground displacements is supported by the use of very high resolution (VHR) optical images.

These techniques can be very useful for relatively slow landslides. For sudden and fast landslides, the use of such information is more problematic. In principle, the prediction might be based on the assessment of well defined indicators to be monitored locally in selected critical points. This approach is somewhere used to predict rock fall through readings of the aperture of joints. In rock masses, a new approach is being investigated by French researchers...
through the analysis of microseismic waves propagating from fractures which are subjected to pre-failure movements (Senfaute et al. 2003). Further experiences bearing on monitoring of precursors and indicators of different types of landslides in various materials are being carried out in some parts of the world (see, for instance, Baum et al. 2005; Flentje et al. 2005).

As shown above, special indicators can be used in the case of rainfall-induced flowslides in unsaturated granular soils. In fact, at C.I.R.I.A.M., flume tests are being carried out to calibrate different sensors to capture indicators of impending failure in pyroclastic soils. At present, TDR probes and optical fibres are being tested.

The TDR technique has been used for decades to measure the volumetric water content of shallow soil layers through a metallic probe which is gently buried in the soil with very small disturbance. The volumetric water content, $\theta$, can be obtained from the strong correlation existing between this and the bulk dielectric permittivity of soil, $\varepsilon_r$ (Campbell 1990). Usually, soil permittivity is estimated by measuring the mean speed of an electromagnetic pulse travelling along the probe, which is correlated to the mean water content in a cylindrical volume of soil around the probe. Several expressions of $\varepsilon_r(\theta)$ relationship, obtained from either empirical correlations (Topp et al. 1980) or semi-analytical approaches (Roth et al. 1990; Whalley 1993; Gong et al. 2003), are available in the literature. However, the retrieval of the entire profile of the volumetric water content along a vertical section can be much more useful in prediction of landslide triggering than the average value obtained from usual correlations. An inverse procedure from TDR measurements has been recently developed by Greco (2006). Such a technique appears very consistent to landslides in pyroclastic soils, which generally present a thickness comprised between some decimetres and a couple of metres; as a consequence, the complete water content profile might be investigated through few long probes driven normally to the ground soil surface. The water content profile can lead to the suction profile through the retention curve of soil. Since TDR probes are very cheap and do not require significant maintenance, site measurement can be carried out with a very low cost. Following the proposed procedure, in the flume, a probe is placed normally to the ground surface, allowing the assessment of the complete profile of the volumetric water content because of the relatively small thickness of the layer (10 cm).

Monitoring of soil deformation through optical fibres is performed by stimulated Brillouin scattering (SBS). This allows measurement of temperature and strain along a single-mode optical fiber. The basic measurement scheme involves the interaction between a pulsed pump beam and a counter-propagating cw (continuous wave) probe beam at a different wavelength. At any section of the sensing fiber, a power transfer between the light pulse and the probe beam occurs, with this interaction being maximum for a precise value of their frequency offset (the so-called Brillouin frequency shift), which in turn depends on the material condition (temperature and strain), and in particular, Brillouin frequency shift increases linearly with temperature and tensile strain. Positional information is obtained through a time-domain analysis; shorter pulses increase the spatial resolution. The key idea is that optical fibres can be laid down in very shallow trenches dug along the slope, in order to capture every soil deformation in the pre-failure stage occurring everywhere. In fact, different from much more expensive instruments, such as extensometers or inclinometers, measurement with optical fibres is not local but distributed. The experiments performed in the flume are carried out by a single-mode standard optical fiber having a total length of about 35 m. Two 1 m long strands are buried in the soil, in the longitudinal section of the
flume. The two strands are separated by a fiber spoil of about 15 meters, placed outside the soil and not subjected to strain.

Some tests have been carried out in the flume using either TDR probes or optical fibres. An example is reported in Figure 5 which reports internal values of suction and vertical displacements of the ground surface (Figure 5a) as well as vertical profiles of the volumetric water content (Figure 5b) in a test performed on a 40° slope made up with the same material used in previous experiments (Figure 2). In this case, the void ratio of soil was 2.35. Once again slope failure is preceded by suction decrease and volumetric compression of soil as a consequence of water infiltration. Regarding the volumetric water content, Figure 5b shows that, at any depth, it continuously increases with time. In particular, in the first stage of the test, the curve \((\theta, z)\) shows a strong curvature since infiltration involves the uppermost layer only. In the successive phases, the water content tends to a uniform value which is attained before failure. A volumetric water content very close to porosity is then attained when suction vanishes and the degree of saturation becomes close to one (Figure 5a). In such a phase, a compaction of the base of the layer seems to occur.

Figure 6 shows the response of the fiber during the test: the two grey zones indicate the position of the two strands of fiber embedded into the soil. The profile recorded 18 minutes after the beginning of the test shows that the only significant change is localized in the fiber spoil comprised between the two embedded regions. As this part of the fiber is not subjected to strain, such variation is to be attributed to the cooling due to water wetting. A successive profile recorded 15 min later shows that the central fiber spoil is still at a lower temperature with respect to the reference profile. Also, a net Brillouin frequency increase is recorded around to a length of 28 m (Figure 6), i.e. in one of the two strands embedded into the soil; a smaller Brillouin frequency shift increase can also be recognized in correspondence of the other embedded strand (at about 12 m), but it is barely visible due to noise. Since no changes in temperature have been measured in the soil layer through thermocouple measurements, the increase in Brillouin frequency can be interpreted as a result of tensile strain induced by soil movement. The final record taken after failure shows that the Brillouin frequency in the embedded regions is going back to the value measured before failure.

In conclusion, the test clearly shows that impending failure is announced by all instruments. Before collapse, suction progressively falls to zero while the soil is subjected to continuing

Figure 5: (a) Records of suction and settlement of the ground surface and (b) assessment of the volumetric water content in a flume test (Damiano et al. 2008)
settlement, then a positive pore pressure (as in Figure 2, but not reported here) is measured at the base of the layer as a consequence of formation of water ponding. In this stage, the water content becomes uniform all over the layer, as shown from elaboration of data provided by the TDR probe. This reveals the usefulness and reliability of this instrument for continuous temporal and spatial information about water content changes. Approaching failure is revealed also by records taken by the SBS-based set-up. In fact, slope collapse occurs 5 minutes later the appearance of strain shown in Figure 6. This confirms the reliability of optical fibres to capture any deformation occurring prior to failure.

Figure 6: Records taken by the SBS-based set up (Damiano et al. 2008)

PREDICTION OF RAINFALL-INDUCED LANDSLIDES IN UNSATURATED GRANULAR SOILS

Foreword
Large part of the world is subjected to catastrophic rainfall-induced landslides. This is particularly true for tropical regions covered by granular geomaterials, as residual soils, which are subjected to intense seasonal precipitations. Based on collection of data on landslides and related triggering rainfall, thresholds often based on a combination of rainfall intensity and duration, have been obtained for several regions, as Hong Kong (Finlay et al. 1997), California (Campbell 1975; Wilson & Wieczorek 1995), New Zealand (Glade et al. 2000) etc. Bearing on empirical data, these approaches can be employed only at a local scale.

In some of these countries, early warning systems have been conceived to prevent disasters. In fact, these thresholds, in combination with rainfall forecasts and real-time rainfall monitoring, can lead to operational landslide warning systems. As an example, in 1977, the Hong Kong Geotechnical Engineering Office established a warning system, which has been continuously updated and improved in the years (Chan et al. 2003). Similar systems have been elaborated to prevent the consequences of rainfall-induced debris-flows in the S. Francisco Bay (Keefer et al. 1987) and in Nagasaki (Yano & Senoo, 1985). d’Orsi et al. (1997) report the Rio-Watch, an alert system based on a network of 30 telemetered rainfall gauges and weather radars which cover the city of Rio de Janeiro which issued 42 warnings between 1998 and 2003. Similar systems have been set up in the State of Oregon (Mills 2002), in UK (Cole & Davis 2002) and in the area between Seattle and Everett, Washington
(Baum et al. 2005). Even though a true early warning system has not been set up, in the landslide prone area around Wollongong, Australia, a monitoring system is active which provides continuous information through the WEB about slope stability conditions (Flentje et al. 2005).

Most of these and other systems are simply based on real-time rainfall monitoring and data transmission. However, there are more sophisticated systems which rely on different indicators through measurement of soil wetness, pore pressures, ground displacements, etc. Analysis of all data is used to establish different levels of warning with shorter and shorter lead times. For instance, to make safe the rail traffic between Seattle and Everett, three warning levels have been established (Baum et al. 2005): the first one, “advisory”, is based on water contents measured in monitored sites; the second one, “watch”, is activated when an empirical rainfall intensity-duration threshold is being exceeded; the third higher level, “warning”, is based also on monitored values of pore pressures.

**Landslide Hazard in Campania Region**

Old chronicles demonstrate that the pyroclastic cohesionless soils which mantle mountains and hills in Campania are periodically subjected to catastrophic rainfall-induced debris flows (Cascini and Ferlisi, 2003). As a consequence of the rapid and extensive development of urban areas and infrastructures occurred after the second World War, the loss of lives and the damages has been exceptional in the last half of the last century (Picarelli et al. 2007). In particular, in the last ten years, debris flows provoked almost 180 victims in ten different sites of Campania, but much higher is the number of slopes which were subjected to landslides bearing no severe consequences. A major catastrophe occurred on May, 5, 1998, when 160 people lost their lives in five different towns: 137 people died in the town of Sarno (Cascini et al. 2000). The last killer landslide occurred in 2006, in the well known island of Ischia, in front of Naples.

This situation is frankly unbearable for an advanced country which should employ a significant part of its gross national product to guarantee the wealth and safety of people. As a matter of fact, in the last years, hundreds of millions of euros have been spent to protect the urban areas subjected to the 1998 debris flows with passive works (Versace et al. 2007). However, since the total number of towns at risk has been evaluated to be higher than 200, a global approach urges, based on sophisticated and possibly cheap systems for risk mitigation. Even with limitations due to the complexity of the procedures and to some uncertainties in the interpretation of data, the use of methods for precocious prediction of impending failure might represent an effective and economic way for risk mitigation. As a matter of fact, early warning systems are being used to prevent further loss of lives in non-protected zones of the area. As other traditional methods, these are based on a comparison between present and antecedent critical rainfall.

The first system has been adopted by the Società Autostrade Meridionali (SAM), which manages the highway between the towns of Napoli and Salerno. The highway runs very close to the coast alongside the bays of Naples and Salerno, at the foot of calcareous relieves covered by pyroclastic soils. In the last fifty years, the highway has been subjected to three landslide events: the first one occurred on December, 8, 1960, when two flowslides were triggered at a time interval of a few hours each other; another flowslide occurred on March, 6, 1972; the third event happened on January, 10, 1997, which caused the death of a car driver. As a consequence of this last event, SAM decided to set up an early warning system whose aim is to stop the traffic when rainfall measured by three automatic rain gauges located near
by of the highway, attains a given threshold. The threshold has been established accounting for the cumulated rainfall over the last 58 days and in the last 24 hours, assuming as a reference the three mentioned landslide events (Fenelli 1998). Reference data concern the period comprised between January, 1951, and September, 1997. The warning signal has been activated twice: by the event of Sarno (May, 5, 1998) provoking no landslides in the vicinity of the highway, and by another event occurred on March, 5, 2005, which caused some victims in the proximity of the highway.

Another model based on similar conceptual considerations has been proposed by Rossi and Chirico (1998) assuming as a reference to the Sarno event. A similar more articulated model (FLaIR) is shortly described in the following section.

**The FLaIR Hydrological Model Used in Sarno and in the Surrounding Areas**

Previous considerations show that prediction of rainfall-induced landslides is mostly carried out through the so-called hydrological models, which are based on historical data regarding landslides and related triggering rainfall (Fukuoka 1980; Mitchue 1985). In Italy, Sirangelo and Versace (1992) proposed the general hydrological model FLaIR (Forecasting of Landslides Induced by Rainfall), which has been recently extended to landslides in pyroclastic soils (Versace et al. 1998, 2003).

FLaIR consists of two modules: RL (Rainfall–Landslide) and RF (Rainfall Forecasting). Through a calibration of available data, the first module correlates precipitations and landslide occurrence, in order to determine a mobility function $Y(t)$. This module enables model calibration and permits the reproduction of historical movements. The second module provides a probabilistic prediction of rainfall events through a stochastic rainfall or meteorological rainfall nowcasting, which is used to identify hazard conditions for landslide occurrence suitably in advance. Using both modules, the model enables a probabilistic evaluation of future landslide occurrence.

In the RL module the mobility function $Y(t)$ which, at any time, depends on the amount of infiltrated water, is associated with the probability $P[E_i]$ of landslide occurrence at the time $t$, by the relation:

$$P[E_i] = g[Y(t)], \text{ where } 0 \leq g(\cdot) \leq 1$$

Among various relationships between the mobility function and the probability of landslide occurrence, a very simple threshold scheme can be assumed:

$$P[E_i] = \begin{cases} 0 & \text{if } Y(t) \leq Y_{cr} \\ 1 & \text{if } Y(t) > Y_{cr} \end{cases}$$

where $Y_{cr}$ is the threshold value of $Y(t)$.

According to [2], which establishes a deterministic relationship between the value assumed by the mobility function and landslide occurrence, the event is certain only if the mobility function $Y(t)$ exceeds the threshold value $Y_{cr}$.

The mobility function $Y(t)$ is defined as:
where \( I(u) \) is the infiltration rate.

In particular, the mobility function can be linked to the infiltration through the expression:

\[
Y(t) = k_0 \int_0^t \psi(t-u) I(u) \, du
\]

[4]

where \( \psi(.) \) is a filter function and \( k_0 \) is a constant depending on the features of the subsoil. It is worth noting that a critical role is played by the choice of the filter function which can model a wide range of situations (Sirangelo & Versace 1996).

The infiltration rate \( I(u) \) is considered to be proportional to the rainfall intensity, \( P(.) \), according to the following relationship:

\[
I(u) = rP_s(u)
\]

\[
P_s(u) = \begin{cases} 
P(u) & \text{when } P(u) \leq P_0 \\
0 & \text{when } P(u) > P_0
\end{cases}
\]

[5]

where \( P_0 \) depends on soil features, and \( r \) is a factor of proportionality. Because the mobility function is defined up to an arbitrary multiplicative factor, it is possible to choose \( rk_0=1 \) so that:

\[
Y(t) = \int_0^t \psi(t-u) P_s(u) \, du
\]

[6]

As shown, the use of FLaIR for real time forecasting of landslide occurrence consists in evaluating the probability that, at time \( t \), the mobility function \( Y(t) \) reaches or exceeds the critical value \( Y_{cr} \), established through historical information about previous slope failures.

The value \( Y_t(t) \) that the mobility function will assume at time \( t \), calculated at the time \( \tau \) \( (\tau < t) \), may be written by splitting the convolution integral [4] in two parts:

\[
Y_t(t) = \int_0^\tau \psi(t-u) P(u) \, du + \int_\tau^t \psi(t-u) P(u) \, du
\]

[7]

The first one, on the right-hand side, is calculated on the basis of observed rainfall. It can be considered as the deterministic component, \( Y_t^{(det)}(t) \), which depends only on the rain fallen in the past and is known at the current time, after model identification and parameter evaluation. The second one is the stochastic component, \( Y_t^{(sto)}(t) \), which depends on the rain which should fall in the interval \( [\tau, t] \), and can be estimated through rainfall forecasting. Therefore, the equation [7] may be expressed in the following form:

\[
Y_t(t) = Y_t^{(det)}(t) + Y_t^{(sto)}(t)
\]

[8]
The division of the mobility function into a deterministic and a stochastic component plays a fundamental role in the procedure of real-time forecasting. In fact, it allows the calculation of the deterministic part of the mobility function through a real-time monitoring of rainfall, so the data uncertainty in the evaluation of $Y_d(t)$ is limited to the stochastic component. As a consequence, the model may be usefully employed to forecast the hazard of rainfall-induced landslide, allowing the activation of the necessary procedures for civil protection. Through the probabilistic forecast of future rainfall, it could be used for the definition of operative rainfall thresholds.

The strategy of civil protection agency with respect to landslide or inundation events is usually based on three warning levels: “attention” (or “advisory”), with instrumental real-time monitoring and real time simulation model running; “alert” (or “watch”), involving civil protection agencies and field direct control; “alarm” (or “warning”), involving population to be evacuated. Using FLaIR, a characteristic mobility ratio $\chi = Y/Y_{cr}$ can be associated with each warning level.

Let $Y_a$ be the mobility function value related to “attention”, (“alert” or “alarm”) threshold, and let $Y_{(0)}^{(0)}(t)$ be the deterministic component, by the condition $Y_d(t) \leq Y_a$, the expression [7] becomes:

$$\int_{\tau}^{t} \psi(t-u) P(u) \, du \leq Y_a - Y_{(0)}^{(0)}(t) \quad \text{with} \quad Y_a \geq Y_{(0)}^{(0)}(t) \quad [9]$$

Therefore, it is straightforward to obtain the rainfall height, $H_{\tau,t}$, cumulated over the time interval $[\tau, t]$ such that the mobility function does not exceed a given threshold value $Y_a$.

FLaIR can be used for warning using only the RL module, so each warning threshold is activated simply when a fixed value of the mobility ratio $\chi$ is exceeded (Sirangelo & Braca 2002), or using also the RF module, thus each warning threshold is activated when the probability of the mobility ratio $\chi = Y/Y_{cr}$ exceeds a fixed value in a fixed forecasting time.

The choice of the values of index $\chi$ for each warning level must fit considering the necessity to have an adequate safety margin, that needs a low mobility ratio, and to avoid false alarms, that needs a high mobility ratio. Values of the mobility ratio used in different applications are the following: $\chi = 0.40$, for the “attention” threshold, $\chi = 0.65$ for the “alert” threshold and $\chi = 0.85$ for the “alarm” threshold.

**Application of FLaIR**

On May, 5th, 1998, after heavy and persistent rainfalls, more than one hundred landslides were triggered in a time span of a few hours along the slopes of the Pizzo d’Alvano mountain, triggering more than one million of cubic meters of material (Cascini et al., 2000). According to several site observations, failure took place in the steepest parts of the slopes, in many cases at the head of gullies, where the top of the bedrock, consisting of fractured limestone, presents a slope ranging 40-45°. The landslides gave rise to forty main debris flows, which ran 2-3 km into the surrounding lowlands, reaching the towns of Sarno, Siano, Bracigliano and Quindici, all located at the foot of the mountain: 159 people died and severe destruction was provoked. During the same event, another person was killed in the town of San Felice a Cancello, which is located relatively far from Pizzo d’Alvano mountain.
In the following years, the area has been protected with passive works; in addition, an early warning system based on FLaIR has been adopted to prevent further victims in non-protected zones. The model has been calibrated using the rainfall data recorded by the Santa Maria La Foce rain gauge, the only instrument present in the area of Sarno (Figure 9), located at an elevation of 36 m above sea level and operated for 66 years.

![Figure 9: Location of the Santa Maria La Foce rain gauge and main debris flows triggered by the 1998 event in the town of Sarno](image)

The model calibration led to the following filter function:

$$\varphi(t) = \omega \beta_1 \exp(-\beta_1 t) + (1-\omega) \beta_2 \exp(-\beta_2 t), \quad \beta_1 \geq \beta_2$$

Such an expression is the result of two negative exponential functions: the first one reproduces the effect of recent rainfall (short term component); the second one, those of the antecedent rainfalls (long term component). The parameter $\omega$ is representative of the relative weight of the two mechanisms in the hill slope behaviour. In particular, for the area around Sarno, it has been assumed:

$$\omega = 0.1; \quad 1/\beta_1 = 0.75 \text{ days} = 18 \text{ h}; \quad 1/\beta_2 = 150 \text{ days} = 3600 \text{ h}; \quad P_0 = 7.5 \text{ mm/h}.$$  

Accounting for the rainfall which led to the event of May, 1998, the threshold value of the mobility function [2] is $Y_{cr} = 9.11$ (Figure 10).
Figure 10: Threshold value of the mobility function matched during the event of May 5, 1998. Referring to the rainfalls records at the Santa Maria La Foce rain gauge in the last 33 years, the mean and minimum time span between the attainment of the “alert” and of the “alarm” thresholds has been 5 and 4 hours respectively which have been considered long enough for all the operations to be carried out in the following phase of “alarm”.

To check the performance of the forewarning system, Figure 11 summarised the data concerning the wet season 2000–2001. It is shown that in the period October 2000 - June 2001, the attention level has been attained several times, the alert level has been reached three times, while the signal of warning has been launched just one time.
Some Considerations about Criteria for Precocious Alerting in Instrumented Areas

Hydrological models can be very useful to predict rainfall-induced landslides in well known geomorphological contexts for which documented data are available. However, a more rational approach might be based on advanced methods based on the analysis of infiltration and of its consequences on the stability of slopes taking into account for both saturated and unsaturated soil conditions and properties (Iverson 2000). As an example, a 2D model based on Richard’s equations under the Freeze’s assumptions (1971) has been developed for real time analysis of slope stability in Sarno (Capparelli 2005). The model, named SUSHI (Saturated and Unsaturated Simulation for Hillslope Instability), consists of two modules: the first one considers the groundwater circulation and consequent pore pressure distribution; the second one, analyses the consequent change in slope stability.

Because of the great uniformity of the geomorphological features of the areas subjected to debris flows (Picarelli et al. 2006, 2008a), numerical approaches could apply very well in Campania. The same reason encourages the use of simplified numerical codes supported by GIS, as SHALSTAB (Montgomery & Dietrich 1994) and TRIGRS (Baum et al. 2002), which integrate, over vast areas, data on rainfall with analysis of consequent infiltration and slope stability. In spite of their still strong limitations, as the use of 1D analysis for both infiltration and stability and the assumption of full soil saturation, these methods represent a major strategy for prediction of rainfall-induced landslides in vast areas for which today weather forecasting can be carried out with some degree of reliability.

C.I.R.A. (the Italian Centre for Aero-Spatial Research) and A.M.R.A. s.c.a.r.l. (Analysis and Monitoring of Environmental Risks) are jointly developing a research program which follows precisely this strategy (Schiano et al. 2007). In order to check the reliability of such an approach, an experimental program will be carried out on small and medium scale physical models (flume tests) reproducing simple representative geomorphological situations, and back analyses will be performed of the behaviour of sample slopes subjected to previous landslides.

A relevant problem is the still poor quality of rainfall forecasting at the scale of single slopes. This does not allow a confident use of early warning procedures based on the analysis of slope behaviour, due to the high probability of false or missing alarms. As a consequence, the analysis should be supported by real-time monitoring of local rainfall and of other fundamental indicators, such as pore pressure. Since accurate analysis is a complex problem because of the difficulty in fixing reliable initial conditions, especially for unsaturated soil (suction), the main advantage of such an approach is that monitoring can provide local values of suction or of water content profiles (which can be related to suction profiles) through TDR probes. Therefore, any numerical simulation can start from a correct initialisation of the governing factors. In addition, the continuous check of these factors can lead to a real-time calibration and consequent adaptation of the model. Finally, as with FLaIR, prediction can be carried out starting from the present (well known) situation, using as input to a meteorological or stochastic rainfall forecasting, or a Bayesian combination of both (Versace 2004). Just to summarise such considerations, an advanced early warning procedure concerning instrumented slopes covered by unsaturated pyroclastic soils should require the following steps:

1. rainfall monitoring and forecasting;
2. start of analysis and models routing;
3. model calibration and adaptation;
Rainfall Monitoring and Forecasting

Naturally, the basic situation corresponds to “normal” weather conditions, which are characterised by absence of rainfall or by “normal” rainfall. In this situation, periodical surveys on site and consequent updating of the geological and geotechnical data should be carried out. In addition, routine checking of all instruments and of the systems to be used for data transmission should be demanded: technical signals regarding malfunctioning of instruments activate maintenance and repairing operations.

An “advisory” signal should be launched when weather forecasting anticipates the approaching of an abnormal rainstorm or when the rain gauge network reports unexpected severe rainfall. Such a signal activates a technical office in charge of all activities concerning analysis and interpretation of data from monitoring.

Start of Analysis and Model Routing

As the advisory signal has been received, local administration is warned about what is happening and can prepare all actions to be activated in case of further warning signals. In particular, purposely appointed people should carry out surveys in the critical sites to collect any additional information to evaluate what is happening and could still happen. A prominent action to be carried out is strengthening of the monitoring in the instrumented sites and analysis of the likely effects of approaching rainfall (which can be roughly estimated through weather forecasting) accounting for monitored values of suction. Numerical models, if allowable, start to run.

Analysis Calibration and Adaptation

Data coming from monitoring enable a continuous and timely check of the analysis by comparison of calculated and monitored values of suction (or of water content) during rainfall. In particular, the data provided by site monitoring of rainfall and suction can be used to update the initial and boundary conditions as well as other parameters which govern the slope behaviour. Hence, the variation of the safety factor of the slope can be continuously adjusted. As a consequence, a framework about what can happen in next hours, i.e. of the presumed scenario of event, can be drawn.

As an example, Figure 12 shows the analysis of suction changes performed with the ABAQUS code and using a well known equation to predict the evaporative flux (Wilson 1990) for the case reported in Figure 3. Accounting for the friction angle of soil (38°) and for slope morphology (which is very close to the one of infinite slope), failure should occur when suction approaches zero. Accordingly, the figure suggests that several times the slope has been rather close to collapse. The changes in the safety factor caused by alternating wet and dry phases are reported in Figure 13.
Figure 12: Comparison between measured and calculated values of suction at a depth of 55 cm for the slope in Figure 5 (Olivares et al. 2003)

Figure 13: Safety factor of the slope in Figure 5 as a function of precipitations, reported in the uppermost part of the figure (Olivares et al. 2003)

**Iteration of Analysis and Prediction**
Adjustment of the analysis by model adaptation allows to improve the quality of the results and more and more rely on prediction. In this phase, an assessment of what can happen in the next x minutes can be carried out through numerical modelling of infiltration and slope stability analysis in the assumption that rainfall will continue with the same intensity, or through stochastic forecasting of continuing rainfall. The process must be iterated until possible activation or deactivation of one of the established levels of warning.
**Decision Making**

The decision depends on the values of the thresholds which have been established. The first step after the advisory signal is the “watch” signal. This should be launched when the time span before “warning” is long enough to perform all the operations required to assure safety; in particular, it should activate a series of actions, such as:

- transmission of data to the municipal office charged for spreading of information and activation of safety measures;
- information to population, which is invited to stay in touch with the authorities to receive further updates about the degree of risk;
- activation of self-protection measures as closure of basement areas, preservation of goods located in area susceptible to invasion of mud, securing of vehicles;
- interruption of activities with which the event could interfere, and securing of machineries;
- possible evacuation of pre-established buildings located in critical areas.

In this phase, indicators of failure as inclinometers, optical fibres or other instruments, and mostly, site survey, might be highly beneficial, supporting next decision.

When the “warning” signal is spread, pre-established actions must be activated, as:

- self-protection measures such as switch off the electric power and secure the gas valve and immediate abandoning buildings located in areas exposed to mud invasion;
- prevention and rescue manoeuvres, mobilization of task force and voluntary organizations;
- security procedures established by the emergency plan (activation of waiting and shelter areas, meeting points…….).

As discussed above, in any phase of the process of landslide prediction the evolution of rainfall intensity and of suction (or pore pressure) must be closely monitored in order to check if prediction matches reality. In particular, any change of rainfall features or any disagreement between prediction and reality must be accounted for. For instance, if in the meantime rainfall intensity declines and suction evolves differently from the prediction, a “normal” condition can be established once again. Therefore, the watch level is crucial because it can lead to warning or to return to an advisory stage depending on the evolution of the meteorological event.

**CONCLUSIONS**

Early warning is becoming a fundamental tool for mitigation of natural risks. However, it cannot be considered a panacea for any type of risk. In fact, it can be very well applied to situations characterized by a lead time which is long enough to activate adequate procedures for risk mitigation. This is possible for intense meteorological events, tsunami and floods involving very large basins. The problem is much heavier for events having a very short lead time, as earthquakes or rapid landslides, which require special procedures to be adopted. In case of landslides, early warning systems are still based on empirical methods relating rainfall intensity with landslide occurrence. Such an approach, which is presently adopted in Sarno and in other areas of the world, can be applied with some confidence only where uniform deposits outcrop and well documented data exist about historic events. In addition, it is subjected to false or even missing alarms. Today, the focus is on more refined procedures based on real-time analysis of slope behaviour, bearing on rainfall forecasting and monitoring. A major problem is, generally, the absence of reliable data about stratigraphy, soil properties
and initial conditions on the vast areas which can experience slope failure. Today, this approach could be more effectively applied to selected instrumented slopes for which data on stratigraphy and soil properties are available. In these areas, monitoring can provide prominent data on initial conditions to be used and continuously updated within a real-time analysis of slope behaviour. Indicators of impending landslide occurrence to collect through cheap and extensive monitoring, as well as continuous site surveying, can be highly beneficial in the crucial phase of decision-making.

REFERENCES


pyroclastic soils of Campania Region based on 'infinite slope' analysis.” Accepted for publication in *Engineering Geology*.


**ACKNOWLEDGEMENTS**

The research (“Individuazione di soglie pluviometriche di criticità per frane distruttive a fini di protezione civile”) is supported by the Ministry for University and Research (Mi.U.R.), P.R.I.N. 2006. Dr Luca Pagano is acknowledged for providing some data used in this report. Other data reported in the paper are the result of a research carried out by A.M.R.A. S.c.a.r.l. with the financial support of GIS-CURARE.
APPLICATION OF TIME DOMAIN REFLECTOMETRY FOR QUALITY CONTROL OF SOIL NAILING WORKS

Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Abstract: Time domain reflectometry (TDR) has been explored as one of the non-destructive tests that have high potential for checking the quality of installed soil nails. Since mid-2004, the Geotechnical Engineering Office has been introducing TDR to its soil nailing works under the Landslip Preventive Measures Programme for pilot use during the independent site audit. Up to September 2007, more than 10,000 TDR tests have been carried out. This paper gives an overview of the technique in assessing the quality of steel soil nails pre-installed with a wire in respect of length and integrity of cement grout. It also describes cases where TDR has identified anomalies and the corresponding follow-up actions.

INTRODUCTION

The soil nailing technique was introduced to Hong Kong in the 1980s. It was first used in Hong Kong as a prescriptive method to provide support to deeply weathered zones in otherwise sound material. This was followed by a few cases where passive anchors or tie-back systems were used. As the soil nailing technique only requires small and mobile construction plants, and a soil-nailed slope is less sensitive to undetected adverse geological features and renders a higher system redundancy than unsupported cuts, the technique has become very popular for slope improvement works in Hong Kong since the mid-1990s. They are commonly installed by drill-and-grout method where soil-nail reinforcement, a steel bar, is inserted into a pre-drilled hole, which is then cement-grouted under gravity or low pressure. To date, more than 50,000 soil nails are installed each year under the Landslip Preventive Measures (LPM) Programme administrated by the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD), Government of the Hong Kong Special Administrative Region. Like other buried works, it is difficult to verify the quality of an installed soil nail. In the context of this paper, the quality of an installed soil nail refers to the as-built length and the integrity of cement grout.

In order to enhance the quality control of soil nailing works, the GEO strengthens site supervision and introduces an independent site audit on soil nail construction works, and, in 2001, began to identify and try out potential non-destructive testing (NDT) methods that could be carried out on installed soil nails. With the help of NDT, the overall picture of the quality of installed soil nails can be built up, which facilitates the identification of the areas for follow-up actions. The non-destructive tests are not to replace good supervision at the time of construction. They are to provide additional assurance and a deterrent against mal-practices. Among the potential NDT methods that have been examined, TDR was the simplest and least expensive (Cheung 2003). In mid-2004, the GEO introduced TDR to its soil nailing works under the LPM Programme for pilot use during the independent site audit. During the course of the pilot use, insight and experience were gained in applying TDR technique for quality control of soil nailing works. This allows further improvement and refinement on the sampling strategy for testing, testing procedure, capability of result interpretation and establishment of follow-up actions to be made. This paper gives an
overview of applying TDR technique in assessing the quality of soil nails.

**PRINCIPLE**

The principle of TDR technique was derived in 1950s from that of radar. Instead of transmitting a 3-D wave front in radar, the electromagnetic wave in the TDR technique is confined in a waveguide (O’Connor & Dowding 1999). TDR is commonly used in the telecommunications industry for identification of discontinuities in transmission lines. In the 1980s, the application of the technique was extended to many other areas such as geotechnology, hydrology, material testing, etc. (Dowding & Huang 1994; Siddiqui et al. 2000; Liu et al. 2002; Lin & Tang 2005). TDR is based on transmitting electromagnetic pulses through a transmission line, which is in the form of coaxial or twin-conductor configuration, and receiving reflections at the locations of discontinuities. By measuring the time for the pulses to travel from the pulse generator to the point of discontinuity, one can determine its location using Eq. [1]:

\[ L = v_p t \]  

where \( L \) is the distance between the pulse generator and the point of discontinuity, and \( t \) is the respective pulse travel time. The pulse propagation velocity, \( v_p \), is related to the electrical properties of the material in the close proximity to the pair of conductors by the following expression (Topp et al. 1980):

\[ v_p = \frac{v_c}{\sqrt{\varepsilon}} \]  

where \( v_c \) is the speed of light in vacuum (3x10^8 m/s) and \( \varepsilon \) is the dielectric constant which measures how a material reacts under a steady-state electric field (for air \( \varepsilon \approx 1 \), for cement grout \( \varepsilon \approx 10 \), for water \( \varepsilon \approx 80 \)).

If a wire is pre-installed alongside a soil-nail reinforcement, which is generally a steel bar, as shown in Figure 1, the configuration becomes analogous to a twin-conductor transmission line and the end of the reinforcement-wire pair becomes a discontinuity. This suggests that TDR can be used to determine the length of installed steel soil nails (Cheung 2003; Cheung & Lo 2005).

![Figure 1: Analogy of a soil nail with pre-installed wire as a twin-conductor transmission line](image-url)
As indicated in Eq. (1), the two key parameters that have to be known for the estimation of soil-nail length are (i) the time for a pulse to travel from the reinforcement head to its end, \( t \), and (ii) the pulse propagation velocity, \( v_p \). Eq. (2) further suggests that the pulse propagation velocity, \( v_p \), along a reinforcement-wire pair in air will be much greater (2 to 3 times) than that in cement grout. Hence, the pulse travel time along a soil nail with voids in grout sleeve will be less than that in a fully grouted soil nail of the same length.

Apart from the effect on pulse propagation velocities, a reflection will be induced whenever an electrical pulse reaches the location of discontinuity in the grouted reinforcement-wire pair (e.g. the end of a soil nail or a void). The magnitude and polarity of the reflection depend on the amount of changes in electrical impedance at the location of discontinuity, which can be expressed in terms of the reflection coefficient, \( \Gamma \) (Hewlett Packard 1998):

\[
\Gamma = \frac{V_r}{V_i} = \frac{Z - Z_o}{Z + Z_o}
\]  

where \( V_r \) is the peak voltage of the reflected pulse, \( V_i \) is the peak voltage of the incident pulse, \( Z \) is the electrical impedance at the point of reflection and \( Z_o \) is the characteristic electrical impedance of the grouted reinforcement-wire pair.

Figure 2 shows a theoretical TDR waveform of a cement grouted reinforcement-wire pair with void section in the middle. There will be reflections at the location of the void as well as the end of the pair. According to Eq. (3), a positive reflection will be returned at the discontinuity when there is an increase in electrical impedance (e.g. reflection 1 at the interface of grout/void and reflection 3 at the end of the pair), whereas a negative reflection will be returned otherwise (e.g. reflection 2 at the void/grout interface). Moreover, the pulse travel time is less than that in the fully grouted pair. In other words, one can in-principle determine the quality of an installed soil nail based on a TDR waveform.

![Diagram of a theoretical TDR waveform of a soil nail with defect in grout sleeve](image)

**Figure 2**: Theoretical TDR waveform of a soil nail with defect in grout sleeve
CONTROL TESTS

Prefabricated Soil Nails of Various Known Lengths
In order to investigate the feasibility of applying TDR technique in the estimation of soil-nail length, TDR tests were conducted on prefabricated soil nails of various known lengths. Figure 3 shows the TDR test results where reflections are returned from the respective soil-nail ends and the time of pulse propagation is found to be proportional to the length of the soil nail. This suggests that TDR technique can be used to determine the length of a soil nail with a pre-installed wire.

![Figure 3: TDR results on soil nails of various known length](image)

Note: All dimensions are in mm.

Legend:
- Reflection from end of soil nail

Effects of Medium Encasing a Soil Nail and the Presence of Couplers on TDR Waveform
One of the fundamental requirements for accepting a non-destructive testing is that it should have a known and consistent basis for the interpretation of test result. As the pulse propagation velocity, \( v_p \), is related to the electrical properties of the material in the close proximity to the pair of conductors (i.e. the soil-nail reinforcement and the pre-installed wire) (see Eq. [2]), it is of paramount importance to investigate the effect of different media in the vicinity of a grouted soil nail on the consistency of the TDR results in respect of the waveform and the pulse propagation velocity.

TDR test was carried out on a 100 mm diameter prefabricated grouted soil nail in air. The test was repeated on the same soil nail embedded in soil. Figure 4 shows the TDR waveforms of the two tests resemble each other and the time for the pulses to reach the soil-nail ends are nearly identical. In other words, the materials beyond the grout sleeve seem to have negligible effect on the TDR results. This suggests that the geology in the vicinity of installed soil nails has no significant effect on the TDR test results.
Steel couplers used for composing soil nails to the required design length may constitute a change in impedance, hence induce a reflection. The TDR waveforms of 136 working soil nails with known locations of couplers over seven sites have been examined. Among the 192 couplers installed at these soil nails, 67% did not return with reflections at their corresponding locations. The remaining 33% where the couplers returned with reflections, the amplitude ranged from 6% to 35% of that of the major reflection at the soil-nail ends. Figure 5 shows the waveforms obtained from two soil nails of the same length, one with a coupler and one without. The reflection returned at the coupler is not significant.

**Prefabricated Soil Nails with Built-in Grout Defects**

Based on the contrast in pulse propagation velocity in air and grout and the occurrence of reflections where there is a change in impedance along the reinforcement-wire pair, TDR
results in-principle can be interpreted to infer the grout integrity of a soil nail. To examine this, TDR tests were conducted on prefabricated soil nails with built-in grout defects of varying void sizes at different locations along the soil nails as shown in Figure 6.

![Figure 6: Configuration of prefabricated soil nails with built-in grout defects](image)

**Void between Grouted Sections**

Following Eq. [3], for soil nails with a void between grouted sections, a positive reflection will be returned when the pulse travels from grout to void followed by a negative one when the pulse travels from void back to grout as shown in Figure 7.

![Figure 7: Theoretical TDR waveform of a soil nail with a void between grouted sections](image)
Figure 8 shows the waveforms recorded from the prefabricated soil nails with a void section near the centre of the soil nail and Figure 9 shows the ones with two void sections. The inferred locations of grout/void interface and soil nail end are also shown in the figures. They were estimated based on the pulse propagation velocity in grout and in air determined from the control soil nails and the grout configuration of the prefabricated soil nails. The waveforms of the prefabricated soil nails closely resemble that in Figure 7. The amplitude of the positive reflection returned at the grout/void interface for these soil nails seems to be related to the size of the void section. For soil nails with significant void size, the amplitude of the reflection returned could exceed 50% of that returned at the end.

Figure 8: TDR waveforms for soil nails with a void section between grouted sections

Figure 9: TDR waveforms for soil nails with two void sections between grouted sections
Void at the Soil Nail End
For a soil nail with a void at its end, when an electric pulse reaches the grout/void interface (where impedance increases), a positive reflection will be returned which is followed by a positive reflection at its end (Figure 10).

Figure 10: Theoretical TDR waveform of a soil nail with a void at its end

The waveforms recorded at three prefabricated soil nails with grout defect at the end are shown in Figure 11. The positive reflection returned at the grout/void interface when superimposed with the reflection at the soil nail end, resulted in a major reflection with a rising front spanning over a longer duration and with a flatter gradient than that of the control.

While the signature pattern in relation to grout defect at soil-nail end may not be easily discernible in the absence of the control soil nail, the shortened travel time of the major reflection can still indicate the presence of anomalies.
Figure 11: TDR waveforms for soil nails with grout defects at the end

**Void at the Soil Nail Head**

The effects of grout defect located at the head of a soil nail are shown in Figure 12 where the reflection returned at the void interface is negative (impedance decreases) followed by a positive reflection at its end.

![Figure 11: TDR waveforms for soil nails with grout defects at the end](image)

![Void at the Soil Nail Head](image)

Figure 12: Theoretical TDR waveform of a soil nail with a void at its head
TDR waveforms recorded at the prefabricated soil nails with grout defect at the head are shown in Figure 13. A reflection is returned at the connection between the TDR testing lead and the soil-nail head where there is a change in impedance. The presence of this reflection may obscure the detection of the negative reflections corresponding to the void/grout interface. In these cases, the shortened travel time of the major reflection can help identify the presence of anomalies.

![Figure 13: TDR waveforms for soil nails with grout defects at the head](image)

These test results indicate that soil nails with significant grout defects will result in shorter TDR-deduced length with some characteristic patterns in the TDR waveform. These patterns depend on the location as well as size of the defects. Hence, when a soil nail is found to have a TDR-deduced length considerably shorter than its design length, the TDR waveform should be examined for any telltale signs. If there are significant reflections preceding the major one with amplitude exceeding 50% of the major reflection, this could indicate that there are defects in the grout sleeve. As shown in the previous section, sometimes, the signature pattern in relation to the grout defect may not be discernible, for example, when the voids are located at the end or the head of the soil nails. In these cases, the nature and causes of the anomalies can be ascertained using other NDT. Example of such application is given in the two case histories in the following sections.

**SOURCES OF UNCERTAINTY**

It is inevitable that uncertainty will be introduced to the estimated soil-nail length through the measurements of the two key parameters in Eq. [1], i.e. the pulse propagation velocity, \( v_p \),
and the pulse propagation time, \( t \). Since the pulse propagation velocity is deduced from the interpreted travel time of a pulse along one or more calibration soil nail(s), the uncertainty arising from the latter source will also affect the former source. In general, the sources of uncertainty can be divided into two categories, viz. test-related and test-unrelated sources. The test-related source includes the uncertainty arising from the built-in error of an instrument and the human uncertainty in the interpretation of test results, while the test-unrelated uncertainty includes the natural variability of the characteristics among the calibration and test soil nails. Figure 14 shows the possible sources of uncertainty of a TDR test.

![Sources of uncertainty of a TDR test](image)

**Figure 14: Sources of uncertainty of a TDR Test**

Uncertainties associated with the TDR test method and the inherent natural variation of quality of soil nails have been identified and assessed (Cheung 2006), to facilitate the determination of the precision limit of the test method (i.e. test-related uncertainty) and the effect of inherent normal variation of soil nails on the test results (i.e. test-unrelated uncertainty). Cheung (2006) shows that the 95% confidence level of the precision limit of a TDR test (i.e. test-related uncertainty) according to BS ISO 5725 (BSI 1994a & b), which is not related to the natural variability in soil nail characteristics, is about ±5%. As the built-in error of most commercially available instruments (see Figure 15) is in the order of 0.5%, the human judgement in the interpretation of TDR results appears to account for most of the test-related uncertainty. In this regard, the subject uncertainty has been minimized through the promulgation of a standard procedure for testing and interpretation of test results. They are available on the CEDD homepage (www.cedd.gov.hk).

Cheung (2006) reported that the test-unrelated uncertainty due to reinforcement size (both diameter and length), the ground conditions, and the presence of couplers appears to be insignificant when compared with other sources of uncertainty such as the variability of grout quality among the calibration and test soil nails. Nevertheless, the 95% confidence level of the overall error in length estimation using TDR is estimated to be about ±9%. This increase
in the uncertainty with respect to test-related uncertainty (±5%) reflects indirectly the possible variability in the characteristics of soil nails within a site and between the sites that have been installed to the current construction practice (e.g. the method of soil-nail installation, variability of grout quality, etc).

![Image of TDR instruments](image)

Figure 15: Some commercially available TDR instruments

**APPLICATION**

**Overview of the Pilot Quality Assurance Programme**

In 2004, GEO implemented a pilot quality assurance programme using TDR technique as part of the independent site audit to check the quality of soil nails with pre-installed wires in LPM sites. The objective of the programme is to experiment the use of TDR to supplement the site supervision in the quality control of soil nailing works. The trial also allows experience to be gained with application of the technique so that it could be further improved. Under the programme, 2% of the soil nails with a minimum of 5 are selected from each LPM site for TDR testing. Apart from the TDR test on the calibration soil nails whose length needs to be known for the determination of the pulse propagation velocity, the tests on other soil nails are conducted and interpreted without prior knowledge of their lengths.

Based on the consideration of the uncertainty of the TDR test method and to balance the number of defective soil nails to be detected against the number of false alarm, an ‘alert limit’ was devised at ±15% of the design length such that if the difference between the TDR-deduced length of a soil nail and its design length exceeds the alert limit, that soil nail is considered as anomalous and follow-up action will be initiated. A short TDR-deduced length could be due to either the as-built length of the reinforcement or the pre-installed wire or both is shorter than their corresponding design length and/or there are substantial defects in the grout sleeve. In addition to the deduced length, the TDR waveforms could also provide some telltale signs of certain anomalies. Thus an anomalous test result can be a short TDR-deduced length or a short TDR-deduced length coupled with an anomalous TDR waveform.
Between July 2004 and September 2007, over 10,000 soil nails at about 850 LPM sites have been tested using TDR. In general, the percentage difference between the TDR-deduced length and design length of most of the tested soil nails does not exceed the alert limit. There are a small number of soil nails (less than 1%) with such difference exceeding the alert limit and further investigation was conducted.

During the course of implementation of the pilot quality assurance programme, an experience-based chain of actions, as shown in Figure 16, gradually evolved. If anomalies are identified during the independent site audit, an investigation on the extent and causes of the anomalies will be conducted. The exact course of the investigation depends on circumstances and each case will have to be determined on its merit. In general, it would include review of test results and construction records, conducting additional TDR tests in the vicinity of the soil nails with anomaly, use of other non-destructive techniques (e.g. Cheung 2003; Cheung & Lo 2005), and in some special circumstances, exhumation of installed soil nails to confirm the nature of anomalies (Cheung et al. 2007).

![Figure 16: General chain of actions evolved from the pilot quality assurance programme](image)

In order to further improve the pilot quality assurance programme, the sampling strategy of TDR testing has recently been revised. Instead of testing 2% of soil nails over the entire construction site, the sampling strategy follows in-principle that recommended by BS 6001
(BSI 1999), where soil nails installed at each LPM site are to be grouped into one or more sample lots and randomly selected for TDR tests. The sampling strategy balances the chance of detecting defective soil nails against the number of false alarm, which in turn has implication on the construction time. In general, the testing frequency has increased when compared with the previous practice. Depending on the size of lot, the minimum testing frequency in each sample lot ranges from 5 to 80 soil nails. Moreover, the representativeness of the pulse propagation velocity in the calibration process is further assured by means of increasing the number of calibration soil nails to a minimum of 3.

**Case Study**

Most of the soil nails with short TDR-deduced length as encountered in pilot quality assurance programme are isolated cases, i.e. anomaly found in only one or two soil nails in a site and additional TDR tests on adjacent nails do not show any anomaly. The use of other NDT methods or exhumation of soil nails to confirm the exact cause may not be justified. Generally, a design review would be carried out assuming the anomalous nail to be not fully functional. The following sections describe two cases where more detailed investigation was conducted to demonstrate how TDR helps identify defective soil nails.

**Case A**

The subject cut slope is about 10 m high and 115 m long with an average slope angle of 50°. The upgrading works comprised 95 soil nails (65 at the northern end and 30 at the southern end). All the soil nails were 7 m long and without couplers. During the independent site audit the TDR-deduced length of five out of ten test soil nails were found to be significantly shorter than their design length, and their TDR waveforms were anomalous. Further TDR testing of the remaining nails at the slope revealed two more soil nails with short TDR-deduced length and anomalous TDR waveforms. The seven soil nails with anomalous TDR test results, namely A13 to A17, B14 and C13, clustered at the northern end of the slope (see Figure 17). The waveforms of these seven soil nails differ from those of the remaining 88 soil nails in two aspects (e.g. Figure 18): (i) the presence of significant local reflections between the major reflection from the soil-nail head and that from the soil-nail end, and (ii) shorter pulse propagation time to the end of these soil nails.

![Figure 17: Layout of soil nails at the northern portion in Case A](image-url)
The waveforms of the seven soil nails (e.g. Figure 18(b)) bear some resemblance of the characteristics of that with grout defects in Figure 2), suggesting the anomalies in these seven soil nails could be related to the presence of voids in the grout sleeves. A review of the site records also indicated that significant grout take was encountered at these locations during the construction of the soil nails.

To supplement the TDR tests, another NDT technique, Electrical Resistance Method (ERM), was carried out. This method measures the electrical resistance between a soil nail and a remote electrode. It makes use of the spatial variation of the electrical resistance of soil nails as an indicator for checking the integrity of grout sleeve. If the measured electrical resistance at a soil nail is found to be significantly different from those of the adjacent soil nails of the same configuration, there is a high probability that the soil nail is anomalous. This method does not require measurement involving the pre-installed wire and hence potential anomalies associated with damaged wire can be ruled out. Details of the method can be found in Cheung & Lo (2005). Figure 19 shows that the soil nails with high electrical resistance determined using ERM match with those with anomalous TDR results, lending support that the anomalies were related to existence of substantial voids in the seven soil nails. Due to the sensitivity limitation of the test, ERM is only capable of detecting significant grout defect in a soil nail (Cheung & Lo 2005). This explains why A17 was not identified as anomalous by the ERM. It also highlights the importance of appreciating the merits and limitations of each NDT when they are used in a complementary manner.

![Typical TDR waveform](image)
(a) Typical TDR waveform

![Anomalous TDR waveform](image)
(b) Anomalous TDR waveform

Figure 18: Typical and anomalous TDR waveforms in Case A
The sizes of voids in the anomalous soil nails were estimated to be in excess of 2 m. Given the sizes of the void were substantial, further investigation was carried out where some sections of the subject soil nails were found to be exposed in the voids (see Figure 20). The seven soil nails were replaced and TDR was used to check the grout integrity of the replacement soil nails both during grouting operation and after installation.

**Figure 19:** Variation of electrical resistance along row A soil nails using ERM

![Graph showing variation of electrical resistance along row A soil nails using ERM](image)

Case B

The subject slope is a 350 m long cut slope with a maximum height of about 60 m separated by up to 6 berms, and with an average slope angle of 45°. The lower batters are formed generally in moderately to slightly weathered rock whereas the upper batters are formed in insitu soils of completely weathered rock. Subsequent to a major failure, the slope was upgraded by means of installation of about 2,000 soil nails and some typical rock slope treatment works on the rock portion (Figure 21).

**Figure 20:** A portion of installed soil nail that was exposed in the void

![Image of a portion of installed soil nail that was exposed in the void](image)
During the independent audit, TDR tests were carried out on 14 soil nails and 4 were found with anomalies. Initial investigation involving additional tests on about 40 soil nails was carried out and the number of anomalous nails increased to 9 (Figure 22). The anomalous soil nails were clustered in an area in the middle batter near the southern end of the slope, and all soil nails in the area were 12 m long. The TDR-deduced length for the anomalous soil nails was about 30% to 80% of the design length. The waveforms of some of these soil nails exhibited characteristics of soil nails with significant grout defects (Figure 23).
To ascertain the possible causes of the anomalous TDR results, other NDTs were carried out. ERM was performed on some of the soil nails tested with TDR at Rows T to U after removal of their concrete soil-nail heads. The results shown in Figure 24 indicated that those soil nails with anomalous TDR results generally have higher electrical resistance than the adjacent soil nails.
The soil nails with anomalous TDR results and high electrical resistance could only be due to short reinforcement or significant grout defects in the grout sleeves, or both. Another non-destructive test, namely Vector Magnetometry Survey (VMS), was also carried out on four soil nails to ascertain their length of the soil-nail reinforcement (Figure 24). VMS involves measurement of the magnetic polarisation manifested by the steel reinforcement bar and records the changes that occur at its end. Further details on VMS are given in Cheung (2003). The VMS results indicated that the length of steel reinforcement bars of the four tested nails were in order.

In light of the above non-destructive test results and the TDR waveforms, the anomalies identified by TDR are concluded to be related to the defects in the cement grout sleeve of the soil nails. TDR testing was further extended to cover all soil nails in the area. In total, 187 soil nails were tested by TDR in this area and 24 soil nails were found with anomalous results and they were subsequently replaced.

CONCLUSIONS
The TDR technique can be an effective tool to supplement site supervision in the quality control of soil nailing works, which cannot be checked easily after construction. While TDR, like any other NDTs, does not give definitive answer to the cause of anomalies, it flags up soil nails that warrant further examination and, coupled with appropriate NDTs, the validity of the TDR tests can be ascertained.

REFERENCES


ACKNOWLEDGEMENTS
This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development of the Government of the Hong Kong Special Administrative Region.
ADVANCES IN REAL-TIME MONITORING OF SLOPE STABILITY

James M. Strout, Elmo DiBiagio and Ralph G. Omlı
Norwegian Geotechnical Institute

Abstract: Instrumentation and monitoring for slope stability have been used for many years for scientific/engineering research and for design verification/construction activities. These traditional applications have driven technology development, and now a wide range of hardware and software is available off-the-shelf for system integration. Monitoring systems are now also being deployed as part of early warning systems, where the data collected is used real-time for risk assessment. The stability state of a slope can be continuously evaluated, and necessary measures to protect life and property are only implemented when the stability state changes and the risk for failure exceeds an acceptable threshold. As a risk mitigation tool, monitoring as part of an early warning system introduces additional needs, including:

- Improved tools for locating and identifying potentially unstable slopes
- Telemetry solutions and power management for remote areas
- Data accessibility and presentation, stability model implementation

This paper explores some of the challenges present in implementing these requirements in a real-time slope stability monitoring system for the purposes of early warning, with emphasis on recent advances in technology and techniques that may be used in these systems.

INTRODUCTION

Landslide monitoring often requires deployment of sensor systems in difficult terrain, often in remote areas. More and more frequently the monitoring systems deployed are more than for scientific research, often they are also deployed as part of a risk reduction/mitigation solution, e.g. early warning systems.

Technology developments over the last decade or so have greatly enhanced the 'toolbox' available for engineers to design efficient and effective monitoring systems. In the arena of early warning systems/risk mitigation, the new innovations in remote sensing has provided a means to identify potential risks using historical satellite images to identify slope movements over large regions. Other remote sensing techniques enable detailed monitoring of large areas, using a grid of measurement points, simplifying the need for identification of a few key measurement locations as required for the deployment of traditional point sensors.

The availability of various data telemetry solutions can simplify the local installation by eliminating the need for cables, as well as simplifying the transmission of data from the slope itself to the decision makers using the data. The use of database solutions combined with either dedicated software or on-demand webpage based graphical user interfaces has greatly enhanced data flow and availability. Finally, advances in fiber optic technology have provided us with the opportunity to deploy high density sensor arrays at relatively low cost.
This article explores some of the latest developments in technology, and presents some of the considerations required to deploy monitoring technology in an early warning context. An example of an early warning system including slope stability monitoring is presented.

REMOTE MEASUREMENT TECHNIQUES

INSAR and PSinSAR
Interferometric Synthetic Aperture Radar (INSAR) is a remote sensing technique using radar satellite images. Sophisticated processing of the radar data is used to produce a narrow effective beam. The data acquisition is not affected by cloud cover, or day/night operation.

The radar data is an image, consisting of pixels where each pixel contains two primary data components:

- Signal intensity, e.g. the amplitude of the radar wave reflected back to the satellite from the Earth's surface. Signal intensity is affected by the material and orientation of the reflective surface.
- Phase, e.g. the position along a single waveform. The phase is a function of the distance along the wave path between the transmitter, reflective surface and the receiver. Changing the distance to the reflective surface will change the phase of the received signal (Figure 1).

![Figure 1: Phase shift and amplitude used for the interferometric interpretation](image)

The phase information contained in two radar images acquired with the same transmitter/receiver geometry can be used to generate maps of surface displacements. Comparison of the phase information between two images produces an interferogram, indicating changes in phase and thus ground movement. Only the ground motion component in line with the radar image is detected, the lateral component will not be detected.
When considering the use of radar images (or any image, including photography) for landslide monitoring the following image parameters must be considered (Kääb et al. 2005):

- **Spatial resolution** - determines what size of objects can be detected within the image, e.g. in satellite images with very high resolution, it is possible to discern items less than 2 to 5m in diameter.
- **Spatial coverage** - what area is covered by the image, the dimensions of a single 'scene' (for example 60 km x 60 km).
- **Temporal resolution** - how often is the data collected, can changes over time be seen?
- **Usability of the data** - is the data appropriate for the application? What is the cost and availability?

The theoretical resolution of changes in distance are one half of the wavelength used, however many factors influence the radar images, such as original topography, the baseline digital terrain model, atmospheric effects, variations in satellite orbits, incidence angles for the images and so on, reducing the practical resolution. For INSAR, the practical accuracy is on centimeter scale. This implies that the average movement of an object at the spatial resolution of the image (for example, a 2.5m x 2.5m block) can be measured at very high resolution (cm scale).

Permanent Scatter Interferometric Synthetic Aperture Radar (PSINSAR) is a processing technique where permanent reflectors (e.g. 'bright spots') in subsequent images provide a stable, point-like behavior for geodetic measurements. A time series of radar images are used to develop interferograms for the points; by increasing the number of images used, statistical methods can be employed, reducing the effects of interference (such as atmospheric variability) to increase the effective resolution to the millimeter scale. The reflectors giving the 'permanent scatters' are typically hard surfaces, e.g. roof tops, parking lots, exposed rock or man-made reflectors installed at control points. An example is given in Figure 2. The PSINSAR technique accordingly functions best where a large number of permanent scatters exist, e.g. in urban areas. In remote areas, it may be necessary to deploy artificial reflectors (Figure 3).

Processing of SAR data to extract topographic deformations, regardless of an INSAR or PSINSAR approach, is a highly complex process and, at this time, is limited to research/commercial organizations specializing in this work, although some software packages for INSAR analysis are available for free (e.g. from USA Jet Propulsion Laboratory and Caltech) or as commercially available software. Processing services are available commercially.

The radar images are generally collected according to programmed missions, although background data is also collected. The result is that data availability for any given location is likely sparse, limiting the use of this technique for active monitoring. However, all historical radar data is available, and if sufficient data is available for a location, the INSAR techniques may be applied to identify 'hot spots', or areas where historically ground subsidence has occurred and are thus locations that may warrant detailed study. An example of this is given in the Usoli Dam example given at the end of this article. In the future, data may be more readily available, and it may be possible to initiate monitoring programs based on regular data images of a location. Data is commercially available.
Figure 2: PSINSAR analysis of the city of Trondheim, Norway. The permanent scatters identified are indicated by dots; yellow shows virtually no vertical deformation, light blue is slight lifting of the surface (<5 mm), whereas blue and purple indicate up to 30 mm/year vertical deformation. The hot spot of settlement indicated at approximately the centre of the figure was previously not identified (NGU/ICG 2006).

Figure 3: Installation of corner reflectors at Åknes (NGU/ICG 2006)
Ground Based in SAR Systems
The disadvantage of satellite based interferometry is the availability of data - radar data is sparse, limiting (excluding) possibility to use satellite data for real-time monitoring or for early warning applications. A second disadvantage is that the measured displacements are limited to the normal component of the vector between the satellite and the measurement point. In unfavorable geometry, for example in steeply sloped areas or where the satellite view angle is shallow, the resolution may be insufficient or the normal component will not coincide with the primary direction of movement of the ground being measured.

However, installation of radar systems at fixed locations on the ground allows the application of the INSAR techniques, where both the view direction and the frequency of the radar images can be controlled. The ground based radar is particularly useful for developing digital elevation models and new applications such as displacement detection of earthquake, land slides and subsidence phenomena have been approached and validated.

An implementation of a ground-based SAR system has been constructed by the European Microwave Signature Laboratory (EMSL). The Linear Synthetic Aperture high-resolution radar (LISA) provides radar images in a selectable range from 500 MHz to 18 GHz. The specifications for LISA give spatial resolutions in the range from centimeters to few meters, and accuracy in displacement measures in the millimeter range, depending on the range between the LISA and the surface measured. Various versions of the LISA system have been constructed (Figure 4).

![LISA system](image)

**Figure 4:** LISA system - rail mounted version for remote locations, trailer version for more accessible sites. Photos from European Union Joint Research Centre IPSC website, http://ipsc.jrc.cec.eu.int.

LIDAR
Light Detection and Ranging (LIDAR) is an optical remote sensing technology used to find range and/or other information of a distant target. The range is determined by measuring the time delay between transmission of a pulse and detection of the reflected signal. The direction of the range vector is determined by measuring vertical and horizontal rotation of the light source. The range vector representing the reflecting surface can be decomposed to \(x, y, z\) coordinates. The measurements are automated, allowing thousands of range vectors \((x, y, z)\) points to be collected each second. LIDAR can be used to develop digital terrain models with very high resolution and small grid size.
LIDAR data can be collected using terrestrial scanners and airborne scanners. The effective resolution will depend on range and positioning of the sensor unit. As an example, NASA's ATM LIDAR achieves 10 to 20cm resolution when operated from an aircraft flying at 400 to 800 meters above ground (NASA 2007). A large part of the uncertainty in the measurement is the positioning of the aircraft (based on GPS and inertial measurements). Significantly higher resolution is obtainable using terrestrial instruments and shorter ranges, for example, a commercially available system by Riegl (Model LMS-Z390i) has specifications indicating up to 2 mm accuracy \((x,y,z)\) at 400m offset (Riegl 2007). Other commercial systems can provide similar specifications.

An example of a terrestrial LIDAR application is given by Slob et al. (2004), where the technique is used to develop a highly accurate 3D point cloud models of a rock outcropping. The data is further processed to develop a virtual 3D surface model, and algorithms applied for automatic identification of discontinuity sets in this virtual model.

Similarly, Bitelli et al. (2004) have applied LIDAR measurements to monitoring of landslide bodies. The approach was multi-temporal monitoring, using 3D digital terrain models collected at time intervals. The models are aligned in a unique reference system, and then digitally 'subtracted' to extract the portions of the model where changes had occurred. This technique provides an explicit indication of surface displacements and morphological changes, at the effective resolution of the DTMs and the numerical techniques for aligning the DTMs in the reference system.

![Figure 5: Application of LIDAR (Slob et al. 2004). Laser scan survey setup in the field, Dave (Belgium). The picture on the right shows a detail of the coloured 3D point cloud of the scanned rock face (note that this is not a photograph).](image)

**Robotic Total Stations**

A total station is a surveying instrument, where a single device combines a theodolite (transit), an electronic distance measuring device and software to process the data. A total station allows measurements of angles and distances between the station and survey points, which can then be converted into any convenient coordinate system. The accuracy of the
measurements depends on the quality of the instrument as well as the distance to the survey point. Measurements are dependent on line of sight, although it is not necessary for a visual fix for the station to function. Practical working distance may be up to several kilometers, but with decreasing accuracy as distance grows.

Robotic total stations include servo motors and control software, allowing the station to target and measure a survey point autonomously. By deploying an array of targets, it is possible to automate the survey process, creating time histories of movements of the individual targets, digital maps or other traditional survey products.

An example of this solution has been deployed in Amsterdam, for monitoring of settlements and displacements as part of the metro construction works (Soldata 2007). The system includes 80 robotic stations and 6500 target prisms for the real time monitoring of 1800 buildings. An average of approximately 12 seconds is required to acquire and measure each target.

A robotic station can be installed to monitor movements of fixed points/targets on a suspected unstable slope. Targets may be fixed at any visible locally stable point (e.g. large rock, on a structure, etc.). Installation of robotic total stations in the field requires protection from weather and exposure, as these are high precision instruments. Enclosures or weather covers may give sufficient protection, but problems with snow fall, ice/frost accumulation, dirt, debris and moisture need to be thoroughly considered. The station should be placed on stable ground to avoid problems with tracking and acquiring of the target prisms.

![Figure 6: Total station and targets (Photos/products by Leica. Other manufacturers available in Norway include Trimble, Nikon, Pentax, Topcon, and Sokkia)](image)

**GPS**

The Global Positioning System (GPS) is based on a network of satellites that transmit precise microwave signals, which a GPS receiver uses to determine its location and precise time. Several readings over time are used to calculate the receiver velocity. GPS receivers installed at key points on an unstable slope would provide point deformations over time.

GPS is an implementation of a Global Satellite Navigation System, and is the only one that is currently fully developed. Other systems are being implemented or are proposed:
▪ GLONASS - Russia's global system, which was fully global, but fell into disrepair, and is now being rebuilt in partnership with India.
▪ Galileo - European system under development
▪ Beidou - regional system in China
▪ COMPASS - Planned expansion of the Beidou system to a global system
▪ IRNSS - proposed regional system in India
▪ QZSS - proposed Japanese regional system

The difficulty with applying GPS measurements is accuracy - it is generally accepted that GPS positioning reliably gives +/- 10 to 15m of practical accuracy, although this accuracy may be significantly better under amenable conditions. The positioning errors are due to several sources, including:

▪ Atmospheric effects
▪ Satellite clock errors
▪ Satellite’s own position error
▪ Multi-path distortion (signal reflection from objects on the ground)
▪ Numerical errors in GPS calculations

An improvement to the accuracy is obtained using the Differential Global Positioning System (DGPS). The first implementation of DGPS was made as navigational aides for maritime traffic, providing better positioning data along coastlines and in waterways.

The DGPS is a network of ground based reference stations, which evaluates the error between the known position of the reference station and the calculated position using GPS, and broadcasts this as a correction factor. The DGPS receiver uses this factor (from several ground stations) to improve the calculated position of the receiver using the GPS satellite data. Positioning accuracy can be greatly improved through the use of both code and carrier phase information in the GPS signals. Kondo et al. (1996) found that relative displacements in the order of 2 cm can be detected in the field using carrier phase data from GPS.

Figure 7: Illustration of DGPS principles (Magellan 2007)

The 'Real Time Kinematic' technique is a process where the GPS signal corrections are transmitted in real time from the reference station to the remote GPS receiver. The temporal
variations in the satellite signal transmission (e.g. atmospheric effects) are corrected on the fly, greatly increasing positioning accuracy of the remote receivers.

There are several systems providing the DGPS corrections, either transmitted from the ground reference stations or transmitted from the satellites. Some of the systems available include:

- United States national DGPS, operated by the US Coast Guard
- Canadian national DGPS
- European Geostationary Navigation Overlay Service
- Japan's MSAS (Multi-Functional Satellite Augmentation System)
- Commercial systems including VERIPOS, StarFire and OmniSTAR
- United States WAAS (Wide Area Augmentation System) operated by the Federal Aviation Administration for civil air navigation

Gili et al. (2000) performed a field study, comparing GPS measurements with the results obtained using traditional techniques (inclinometers and wire extensometers) checked against fixed points. The precision they achieved with the GPS measurements is 12 to 16 mm in the horizontal plane and 18 to 24 mm in elevation.

Khamarrul & Wan Aziz (2003) achieved less precision; however they noted that the location had poor satellite geometry as well as relatively long distance to the reference stations. An important comment is that it is not necessary to rely on the national reference stations and beacons, precision can be improved using local references established for a project.

TELEMETRY SOLUTIONS AND POWER TRANSMISSION

Mobile Telephones and Data Transfer

Mobile telephone technology provides for simple and reliable data telemetry from remote sites where mobile telephone coverage is available. Modems are widely available commercially; these may be as simple as a no-frills modem, or may include local micro controllers, data logging and power supply. The modems provide transparent communication for a serial device providing a digital signal.

Evolution of the mobile communications services is yielding increasing data rates as new standards are being introduced:

- GSM: Global system for mobile communication (38.4 kB/s)
- GPRS: General Packet Radio Service (170 kB/s)
- EDGE: Enhanced Data rates for GSM Evolution (236 kB/s)
- 3G: Third Generation Mobile Communication (384 kB/s)

The initial modems for GSM functioned exactly like a dial-up modem - that charges for the service were based on the connection time. For continuous monitoring, this is an expensive approach. In subsequent generations (GPRS and later), the amount of data transferred, and not connection time, is primarily the basis for the charges.

A vast selection of stand-alone modems is available supporting GSM/GPRS, and most manufacturers are also now supporting EDGE. 3G modems primarily directed towards
personal computing are also widely available. Modems may be connected directly to individual sensors, providing distributed telecom data links (eliminating cabling) or as a single modem installed on the system data logger.

For most landslide monitoring applications using traditional sensors, the data capacity provided by a single GPRS is probably sufficient; however if dynamic measurements or for example seismographs are used, it may be necessary to support higher data rates.

Similar to mobile telephone technology are license free radio modems, which provide a point-to-point link for transmitting data. The range available depends on local regulations regarding transmitter strength; in Norway, a license-free radio transmitter will reach 3 km in favorable terrain and conditions.

**Networks**

For our purposes, a network is an interconnection of equipment where communication between the units is managed using some form of standard protocol, and the communication is conducted either via cable or a wireless solution. There are many different approaches available; the ones that are most commonly known include:

- Ethernet: A cabled solution using twisted pairs, for example an office computer network
- Bluetooth: A radio solution, implemented in low cost, small radio chips which can be plugged into other devices
- WiFi: Also a radio solution, this is the most familiar implementation of our home wireless local area networks for our computers

Networks are generally described depending on scale:

- Personal - a few meters (for example the network existing between your wireless printer, telephone headset, telephone and PC)
- Local area - from tens to some hundreds of meters (e.g. your wireless computer network)
- Regional - hundreds of meters to kilometers (e.g. university-wide or city-wide wireless computer networks)
- Wide area networks - kilometers to global scale

For landslide applications, a local area or regional network would probably be sufficient. Larger networks may be built as a collection of smaller networks, for example many local area wireless networks might be interconnected, to give a regional network covering a university campus.

There are many other solutions available, for example bus solutions (such as CANBus) which are prevalent for example in industrial processing or the automotive industry but are not that commonly known for civil engineering applications.

Sensor and monitoring system technology is now widely embracing the use of digital communication and network philosophy, and the implementation of industrial digital standards in the sensors themselves is commonly available. The use of digital standards such as RS485 or TCP/IP allows multiple pieces of equipment to be installed on single cable runs, with shared wires for power supply and signal transmission, and individually addressed sensors and components. This is a great advantage over non-network solutions, where individual cable runs are required for each sensor.
For example, a simple wired network can be established using a data logging system with a serial port, a single cable run for power and digital signals, and intelligent sensors supporting RS485 and addressing. This would replace a central data logger with individual cable runs to every sensor location (Note though that if high speed data measurements are required, e.g. for dynamic measurements, then a more complex solution may be required.).

The complexity of a wired network can be reduced even more, if local power (e.g. batteries) are provided at each sensor and the cable run is replaced by a wireless network link.

**Meteor Burst Communication**

Satellite data transmission is accessible from most locations on earth; however this may be a relatively expensive data link. An alternative (and low cost) solution for modest data sets from extremely remote locations is the use of meteor vaporization trails as a reflector for radio waves.

The Earth’s atmosphere is constantly bombarded by thousands of particles (meteors); and they enter the atmosphere and burn, creating trails of ionized particles. These trails last only for fractions of seconds, but are sufficiently dense to reflect radio waves while they exist.

The window of opportunity for transmitting data is short. In addition, a number of factors influence the distance over which communication can be established, including angle and length of the particle trail, altitude of the trail and the relative positions of the transmitter and receiver.

The Natural Resources Conservation Service of the United States Department of Agriculture (NRCS/USDA) uses meteor scatter to collect data in the SNOTEL system. The system consists of 730 sites in 11 western US states, including Alaska. The installations are primarily in remote areas where access is difficult.

The SNOTEL sites are designed to operate autonomously, requiring only a yearly maintenance visit. Solar cells recharge battery packs for power. Measurements usually include air temperature, precipitation and snow accumulation at 15-minute intervals. Statistics and complete data sets are transmitted using meteor burst communication on a daily basis, although data can be collected more frequently if needed. The newest sites deployed are being configured to allow hourly data collection. The data set is transmitted to 2 master stations operated by NRCS in Boise, Idaho, and Ogden, Utah.

The NRCS reports that a system-wide poll of all stations produces better than 95 percent response from the stations, and up to 99 percent response is common. The high response rate indicates the effectiveness of the meteor burst communication approach.

All system information given above is cited from the SNOTEL fact sheet (NRCS 2007).

**EM Data Transmission (Wireless Underground Sensors)**

Sensors and communication systems are usually equipped with cables for power and signals. There are a number of shortcomings with cable, (after Kogo et al. 2006):

- Cable trenches are weak points, both due to lack of compaction but also as leakage paths for water. In boreholes this may disturb measurement of pore pressures due to vertical leakage.
- Cable failure due to insulation deterioration, damage during construction, or as attractive points for lightning strikes damaging system components and sensors.
- Interference with construction activities: everyone knows that contractors dislike instrument cables, they get in the way of normal construction activity and require special care and handling.
- Cables costs become large as cable lengths increase.

In difficult terrain it may be an advantage to avoid cabling to reduce practical problems related to installation and protection of the cables.

Sensors with wireless signal transmission reduce some of these problems. Sensors installed above ground can easily be adapted to use batteries and to transmit signals via various telemetry solutions (see earlier discussions about networks and mobile telephones). The problem arises when the sensors are to be installed below ground, in boreholes, embankments or fills, requiring transmission of data over relatively large distances though soil or rock.

An approach using low frequency electromagnetic waves allows data to be transmitted through earth and rock, similar to radio transmission through air. Although attenuation is very high, it is possible to transmit data over a limited distance. The principles and an example of application are shown in Figures 8 and 9.

![Figure 8: Principles for EM transmitting sensors (Kogo et al. 2006)](image)

Kogo et al. (2006) have deployed EM-based, battery powered wireless pore pressure transmitters in an embankment (Figure 10). The sensor specifications were:

- Transmission distance: minimum 100m in soil
- Transmission frequency: 8.5 kHz
- Data collection/transmission rate: 1 per day / 1 per week
- Plastic housing (fiber reinforced plastic)
- Dimensions: Ø125mm, height 205 mm.
- Ambient pressure rating: 3 MPa (30 bar)
- Battery life: 10 years

Figure 9: Example of application of EM transmitting sensors

Doe & Enachescu (2007) present the electromagnetic permanent gauge (EPG), a pore pressure measurement gauge developed for the Bure underground research site. The gauge is an adaptation of a tool originally developed for oilfield use. The specifications of the EPG are given by Doe & Enachescu (2007) as:

- Sensor range: 15 MPa and 125 °C
- Accuracy: 1.5 kPa and 0.5 °C
- Resolution: 0.15 kPa and 0.05 °C
- Drift: < 3 kPa and 0.5 °C in 3 years
- Operational life: 4 years (lithium batteries) at 1 reading per day
- Available measurement frequencies: 5/hr, 1/hr, 1/day and 1/week
- Bidirectional communication (allowing changes in measurement frequency)
- Dimensions: not specified, but likely similar to the Kogo et al. (2006) sensor

Transmission distance through soil or rock is limited by the attenuation of the signal, and the capability of the receiver to detect the signal. Transmission distance is greatly enhanced by using waveguides to 'focus' the signal along the path of the wave guide. In the Bure installations, the EPG was installed at the base of a cased boring at depths up to 450m.

NGI has developed an EM gauge for temperature measurements on offshore pipelines (Figure 11). Using the pipeline as a waveguide permits data transmission over many kilometers, from a fixed platform or land to a remote installation. As subsea instrument cables can cost well in excess of 30 USD/meter, cost savings are substantial.

Figure 11: NGI EM temperature sensor. Left: Antennas and electronics pods. Right: installation of pipeline; EM sensor covered by protective casing.

Inductive Power / Signal Transmission Without Enclosure Penetration
Connectors or penetrations through barriers and enclosures introduce a weak leak in equipment where a sealed environment is required, for example sensors installed underwater. Penetrations in the enclosure are prone to leakage, either immediately (if the seal is not adequate) or over time if the seal degrades either through corrosion or material degradation. Temperature and pressure changes can also cause seals to fail due to mechanical and thermal deformations of the enclosures.

A solution to this is the transmission of signal and power through the intact barrier, e.g. through the wall of the enclosure. Induction is the transfer of energy without physical contact using a magnetic field. There are many familiar applications already: your electric toothbrush or personal shaver may rely on an inductive link for recharging the battery, and your modern cooking range uses induction principles to heat your frying pan.

In addition to energy, a data signal can be transferred, for example by modulation of the magnetic field. The transmission range for the commercially available products is generally a
couple of centimeters, although one commercial supplier claims to have developed a solution for low power transmission (not data) with a range up to several meters.

Applications of inductive coupling may be considered where it is imperative to have a fully sealed enclosure, or where it is impractical to have cabling directly to the sensor. As an example: Pore pressure sensors with an inductive link may be installed in the annular space of a grouted-in-place inclinometer casing, eliminating cabling and potential problems with vertical sealing, as well as giving a dual-purpose borehole. Periodic measurements of pore pressures can be made simultaneously with inclinometer profiling, using an inductive link installed on the inclinometer instrument cable (Figure 13).
FIBER OPTIC SENSORS
An optical fiber is a glass or plastic fiber designed to guide light along its length. Optical fibers can be used to measure strain, and indirectly can also measure pressure, rotation, chemical concentrations and other physical parameters. The measurement is based on a modulation of optical characteristics at the point of interest (Figure 14).

![Basic Schematic Diagram of an Optical Fiber Sensor](image)

Figure 14: Optical fiber sensor (Illustration from Smartec website, www.smartec.ch)

The measurements can either be made using the fiber as a continuous distributed sensor (all points along the full length of the fiber), or as measurement at points (Bragg gratings) etched in the fiber at specific locations (Figure 15). Applications of the fiber sensors are the same as those of traditional sensors, with the advantages that multiple measurement points along a single element offer.

A challenge with optical fiber sensors is that the strain measured is a coupled strain, consisting of mechanical and thermal effects. Temperature must always be measured together with other parameters of interest to compensate for the thermal component.

Straining the fiber alters its optical characteristics, which can be measured using a laser beam sent through optical fiber. When measuring other parameters, it is necessary to induce a strain in the fiber as a function of the parameter to be measured. For example, hydrogen gas concentration can be measured by coating the fiber with a coating of palladium, which reacts when exposed to hydrogen resulting in strain in the fiber. The induced strain is measured, and subsequently correlated to the gas concentration.

The strain in the fiber can be measured in several ways, depending on whether distributed or point sensors are employed.

For distributed fiber sensors, no specific points are defined as measurement points; any position along the fiber is a potential sensor. Light traveling through the fiber is reflected at positions where there are changes in the optical characteristics. This change creates a scattering effect, which is measured either using an interferometer (Brillouin scattering) or grating spectrometer (Raman scattering). The travel time for the scattered light indicates the
distance along the fiber to the point where the optical properties were changed (e.g. the fiber was strained).

The spatial resolution of the measurement points in a distributed sensor fiber depends on the accuracy and resolution of the travel time measurement. NGI has deployed a commercially available distributed sensor system for strain measurements in bridge girders, where the system achieves a 10cm separation between individual measurement points along a single fiber. In this system, 5km of fiber optic cable is used, yielding 50,000 measurement points.

Bragg gratings create an optical disturbance in the fiber causing a specific wavelength of light to be reflected. Strain in the fiber changes the separation in the gratings, causing a shift in the reflected wavelength which is directly correlated to the strain. By tuning each grating to specific wavelengths, many measurement points can be incorporated in a single fiber.

Figure 15: Bragg grating (Illustration from Wikipedia)

Optical fibers have several advantages over traditional sensors:

- Small size (diameter)
- No power is needed at the measurement point
- A fiber can either be deployed as a continuous sensor or numerous measurement points can be encoded on the fiber
- The fiber itself is low cost

The instruments required to measure the strain in the optical fibers are relatively costly, with the instruments for the distributed sensors being more costly than for the point sensors. However, the overall cost of the fiber optic system will likely fall under that of a traditional sensors when large numbers of sensors are required.

DATA ACCESSIBILITY AND PRESENTATION
A key point in development in the latest years is the availability and accessibility of data to the end user. Principally the approach is to utilize a system for collecting and organizing the data (often a database), and presentation via a graphical user interface. An example using a dedicated system is given in Figure 16.

By using a database in conjunction with an internet web page, it allows data and other information to be delivered based on specific requests or as information is updated. These database-to-web solutions are supporting increasingly better graphical user interface (GUI), allowing an intuitive means to access and interpret the data. A general model for this is some sort of 'middleware' managing data access and dynamic web page generation. This 'middleware' may range from a custom designed solution for a specific application to an implementation of a commercial software system.
There are some general requirements which should be considered when selecting/designing the ‘middleware’ and the overall GUI:
- Accessibility: Who needs to have access to the data? and who should not have access? (e.g. limited group, or is the data public information?)
- Control: Is the data presented quality controlled/verified?
- Plotting: What plots are required? How should the data be presented?
- Programming: What data model and language should be used? and which platform? Programmed from scratch, or derived via commercially available software?
- Data stream: Handling/collection of new data from the remote site, as well as how to integrate existing data. Browsing of data from the database, for example plotting selected time periods, or combining multiple data sets in single plots. Exporting of data for further analysis
- Provisions in the system for adding new sensors, new data. Flexibility to incorporate new plots in the dynamic web pages.
- Implementation of more advanced modeling or decision making in the 'middleware'
- Robust solutions to handle erroneous data arising from sensor failure (or sensor removal). Plotting/presentation routines to handle missing or erroneous data.

Once the technical solution is in place for accessing, processing and plotting data, the specific details of designing a useable and attractive presentation solution can be made. An example of a functional and attractive GUI is shown in Figure 18.

![Website for real-time landslide monitoring](image)

**Figure 18:** Website for real-time landslide monitoring (Pennell et al. 2005)
EARLY WARNING

Introduction
In common usage, an Early Warning System (EWS) is a risk management system for detecting and dealing with an anticipated natural or man-made hazard. However, early warning systems are not restricted to disasters and hazards; they are applicable to any activity or situation that may create a problem that must be dealt with. A broader definition may be used: Early warning system – a system or procedure designed to warn of an impending problem, in sufficient time to allow for actions to limit the impact of the problem.

There are early warning systems for almost everything conceivable: political unrest, disease, draught, financial loss, rocket attacks, fraud, biological terror, locust swarms, mad cow disease and of course natural hazards. The systems represent a tremendous range of scale, from global (e.g. air and sea temperature changes, global warming), regional (Pacific Ocean tsunamis), local (WWII air raids over London), personal (smoke detector in your bedroom).

An early warning system will normally have a minimum of 5 components, and is implemented in a conceptually simple flowchart model (Figure 20):

1. Knowledge of and means of forecasting the risk faced
2. Information from technical monitoring and visual observations
3. A response plan
4. Dissemination of meaningful warnings to those at risk
5. Public awareness and preparedness to respond to the warning

Figure 19: Events and consequences - why an early warning system may be needed
Figure 20: Basic flow chart for a conceptual early warning system

Geotechnical engineers have always relied extensively on the concept “early warning” in their work, but under another name, i.e. the “Observational Method”. An example of a geotechnical EWS: Braced excavations (Figure 20) using the ‘observational method’.

- Monitoring: The force in the bracing strut is measured
- Analysis /Forecasting: Measured value is compared to limit design value
- Warning of a problem: If the force approaches the design value, a warning is issued to the designer or contractor
- Response to the problem: Add an extra strut

Figure 21: A braced excavation and the ‘observational method’ early warning system
Constructing a System

What to Measure
The key to a successful EWS is to be able to identify and measure the relevant precursors (indicator variables) to the event which the system provides warning for. This requires that the physical system is well understood, and either quantitative or qualitative measurements can be made which indicate how the changes in the physical system affect risk.

For example, some possible precursors for a landslide hazard are:

- Intense rainfall
- Earthquakes and ground vibrations
- High rate of movement of a slope
- Rapid increases in pore water pressure
- Erosion at the toe of the slope

Choosing the parameters to measure will depend on which parameters are the most reliable for indicating changes in stability, and which can also be reliably measured.

For example, in a specific slope it may be possible to identify that changes in pore pressure in a critical layer is the predominant trigger for stability problems, and measurement of pore pressure using piezometers would be the most relevant precursor variable to measure. Early warning would be issued based on these measurements.

However, at another location, it may be more appropriate to examine historical data for rainfall duration/intensity for a region, and base risk assessment on statistics correlating increased landslide activity to rainfall events. Early warning may be issued based solely on weather data and weather predictions.

How to Measure
Once the precursors are identified, an appropriate system for measuring, presenting and interpreting the data is required. Measuring and presenting data presents a lesser problem today, as available technology for this is now highly sophisticated and most system components are 'off the shelf’ and communication interfaces standardized. An integrated system of sensors, power supply, data logging and presentation in a graphical user interface via web can be commercially supplied.

Interpreting the Data
As data is collected, it is necessary to interpret the data real time, or if the measured process is slow at planned intervals (not real time) to allow effective warnings to be issued. This is of course dependent on the system: an early warning system for agricultural crop failure may need data interpretation only weekly or monthly, whereas a warning system for structural collapse of a city bridge may need continuous data interpretation.

Compounding the complexity of the data interpretation is that the data set and the physical model may be either deterministic or stochastic or a combination of both. A deterministic system has a direct relationship between cause and effect, whereas a stochastic system is characterized by randomness.
Interpretation models may be grouped in three general categories (Brecke 2000):

- Sequential models
- Correlational models
- Conjunctural models

The following definitions of these models are adapted from Brecke (2000).

A sequential model consists of a mechanism that describes how changes in a set of deterministic variables bring about a specific event (or events). This is the most common line of thinking for traditional engineering, e.g., a sequential slope stability model depends on geometry, material properties (including pore pressure) and external loading. Variations in these variables lead to predictable changes in stability.

Correlational models consist of relationships between stochastic variables, for example length of rainfall at certain intensity and the length of the preceding dry period are correlated to increasing instability probability in regional slopes. In effect, this model indicates that the engineer identifies that a relationship exists between variables, but the underlying physical process causing the correlation is either indirect or unknown.

The conjunctural model approach focuses on combinations of conditions and events, where different combinations of variables lead to different outcomes. This approach may be most suitable for large variable sets. For example, a bridge monitoring system deployed by NGI utilizes over 50000 fiber optic strain gauges. A conjunctural model approach may indicate that changes observed in strain gauges at various points in the bridge support girders may indicate undetected stress in a completely separate bearing element.

Sequential and correlational models are likely the most accessible and easily implemented as rule-based or case-based systems. In a rule-based system, test criteria are established for specific actions or interpretations, most familiar for engineers as an ‘IF-THEN’ rule set is:

IF pore pressure > 120 kPa, THEN factor of safety < 1, ELSE factor of safety > 1.

Case-based systems function by comparing a collection of data/observations to some form of database of historical measurements, to attempt to identify earlier known situations with similar patterns and known outcome to the current situation.

The implementation of an EWS data analysis model may also consider uncertainty, by introducing probability or fuzzy logic (EPA 2005).

Probability analysis and fuzzy logic are two different approaches to deal with inherent uncertainty in data and analytical models. This is a complex subject and is not discussed in detail; however a simple definition is given to help identify the distinction between the two:

- A probabilistic approach assigns uncertainty by giving a likelihood that a variable is a member of a set; whereas fuzzy logic assigns uncertainty through assigning a degree of membership to a variable in a set. These are expressed via probability theory or possibility theory; two simple examples are given for clarity:

  - Probability: it is 75% certain the pressure is 100 kPa
Possibility: 100 kPa is likely a high pressure (inferring that it might also be low, medium or very high pressure in other reference frames)

Approaches employing advanced statistical/stochastical modeling of large data sets, or the applications of probability/fuzzy logic to early warning systems for geotechnical and civil engineering is still in the research arena, see for example the work by Medina-Cetina and Nadim (2007).

Setting Alarms
The interpretation of the data in the context of the physical system model, and the establishment of threshold values for triggering alarms is the most difficult problem in designing an EWS. Alarms must be triggered early enough to permit actions to be taken, but not too early, and the alarms must be reliable. The consequences of false alarms are often so serious (loss of credibility) that every possible action must be taken to avoid them. The consequences of not issuing an alarm when needed are even greater.

Setting alarm threshold values may be difficult in the initial system design. Some of the problems may include:

- Limited understanding of the physical system (problems with developing a sequential model), or difficulty in identifying clear correlations between measurable variables (problems with developing a correlational model).
- Identifying the necessary indicators, without eliminating important information about processes that we do not yet understand. Technology and costs can bias which indicators we choose to measure.
- Lack of historical/experience data
- Selecting correct level of data resolution, frequency and complexity to capture significant changes in the system.
- Variable system states (e.g. a slope covered in dense vegetation in early summer, may have different threshold values for rainfall compared to the same slope in late fall, when most of the foliage is gone).
- Political/local authority hesitation, reluctant to take decisions

Tuning of the system may be necessary, and experience with data collection and system response may be needed to adjust alarm thresholds to suitable levels.

Once an alarm threshold is exceeded, some form of dissemination of information is required. The consideration here needs to include when an alarm should be issued (thresholds), how it should be sent (the technology used) as well as the content of the message (what is the warning, and how will it be understood).

A system may be implemented with various types of alarms and thresholds, where each initiates a specific action. For example, a system may be developed using several alarm types and response thresholds (see Figure 22).

Each application needs to be individually considered to design an appropriate alarm philosophy, accounting for the nature of the system monitored as well as the needs of the population to be protected.
Sounding the Alarm - The Social Component

Once an alarm situation is reached, some form of message must be transmitted to the authorities or groups identified as necessary recipients of the message.

| Alarm type 1: System component failure | Level 1: Minor, repair at next routine maintenance | Level 2: Essential, repair as soon as possible | Level 3: Critical system failure, initiate contingency plan |
| Alarm type 2: Inconsistent/conflicting data | Level 1: Data does not indicate a potential threat, initiate review | Level 2: Data indicates a potential threat, initiate contingency plan |
| Alarm type 3: Consistent data indicates elevated risk | Level 1: Increased awareness | Level 2: Contact and inform emergency services | Level 3: Initiate mitigation measures |
| Alarm type 0: All clear signal |

Figure 22: An example of how a multilevel/situation alarm scheme may be planned

The technology available for transmitting messages must be adapted to the needs of the EWS and the capabilities of the recipients. For example, relying on instant messaging by mobile phone directly to a population has little value, if the population is poor and only a few can afford mobile phones or if mobile coverage is sporadic or not available. Audible warnings (sirens) may be inappropriate if these will lead to panic and inappropriate actions.

Strategies for issuing alarms can be situation dependent. For example, an alarm representing a system component failure may be transmitted to a technician by SMS; whereas an alarm representing a threat may be sent simultaneously via an audible alarm, telephone to public officials and by radio to emergency response authorities.

Early warning systems are more than just an implementation of technology - social factors must also be considered, e.g. how will the recipient respond to an alarm or warning. Is the response well planned? Are the recipients of the alarm well informed? Will the response be appropriate? Are the public authorities/emergency response authorities sufficiently prepared? If the human and social factors are not considered, the alarm can in a worst case scenario create a new emergency - an example is the evacuation of New Orleans for hurricane Katrina.

Operating the System

Successful operation of a monitoring system, particularly a system for early warning, requires several elements to be considered and put in place:

- Authority and responsibility: Who will operate and care for the system, evaluate data, issue alarms? Who has the authority to activate emergency response plans, for example initiate evacuation?
- Maintenance and test plan: Instruments and technology require maintenance, repairs and possibly upgrading. The system must be routinely tested to verify correct operation, and where possible system element failures should be self-detected in the system (and a warning issued indicating maintenance is required)
- Emergency response training: Routine review of emergency response plans, and training of emergency services.
- Contingency: What will be done if the system fails, or critical sensors are lost? If a series of events occurs (e.g. several landslides), will initial damage to part of the system in an early event prevent other hazards from being detected?

**CASE EXAMPLE: USOI DAM EWS**
The Usoi embankment dam is a gigantic natural structure formed by landslide; it is the largest embankment dam in the world: 600 m high and 5 km long. The dam forms Lake Sarez, which has an average width of 1.3 km and maximum depth of 500 m containing approximately 17 cubic-kilometers of water (Figure 23). If the dam were to fail, the resulting flood would be a catastrophe of unbelievable dimensions! Flood waters would flow down the Bartang valley to the Panj River valley and end up in the Aral Sea.

![Figure 23: Lake Sarez, the Usoi dam and the Bartang valley downstream of the dam](image-url)
The area is subject to seismic activity, and slopes surrounding Lake Sarez are known to have stability problems. The potential risk is a seismically induced landslide/rockfall into the lake, or submarine slides beneath the surface of the lake. The direct consequences to the dam include earthquake damage or internal piping leading to dam failure, and the indirect consequences for the dam are a sudden increase in level of the lake or the generation of surface wave causing overtopping due to slope instability elsewhere on Lake Sarez.

NGI's role has been to participate in the expert group advising the implementation of the monitoring and early warning system. The sensors for the monitoring system implemented include:

- Surface displacements
- Seismic data
- Lake level
- Flow in the Murgab river
- Turbidity
- Meteorological data

The data is transmitted from this remote region to data operations centre by satellite transmission (Figure 24).

![Diagram of EWS components at Lake Sarez/Usoi dam](image-url)  
**Figure 24:** Implementation of EWS components at Lake Sarez/Usoi dam (basis for illustration by Stucky Ltd., redrawn by the authors)

As part of the initial studies of slope stability risk assessment, a PSinSAR analysis was performed on historical satellite data available for this location. The analysis was performed by a commercial organization offering these services. The analysis focused on the right
(eastern) bank of Lake Sarez, identifying an area showing historical movement and allowing
detailed local sensors and instrumentation to be installed to follow future movements of this
slope in the context of the EWS. The analysis is shown in Figure 25.

Figure 25: Permanent scatter interferometry applied to historical data from Lake
Sarez right bank landslide (Image from TRE 2006)

The biggest challenges experienced at this location have not been related to the design of the
technical system or the selection of parameters for measurement. Rather the challenges have
been the practical issues related to installation and operation:

▪ Remote site (access by helicopter/donkey)
▪ Minimal local technical capacity (initial installation, maintenance of equipment)
▪ Poor local population (equipment, supplies, fuel, etc. used for other purposes)
▪ Information (purpose, need and importance of system not well understood by local
  population)

The system is now in operation following several years of planning and installation works,
but is presenting considerable challenges related to maintenance, repairs and continuous
operation.

CLOSING
Developments in technology in recent years have added a wealth of tools and equipment for
monitoring of landslides as well as other geotechnical and civil engineering structures. Advances in remote sensing provide new tools for identification and monitoring of landslides
on a regional as well as local scale. Imaging data from satellite, aerial or even ground-based
combined with interpretation techniques like permanent scatter interferometry provide unique
opportunities for identifying potentially unstable slopes requiring detailed study. Combining
digital terrain models developed using LIDAR with continuous deformation monitoring by
robotic total stations or GPS installations provide exciting ways to monitor detailed changes
in slope morphology via continuous monitoring.

Data communication from monitoring systems has been revolutionized by the availability of
mobile data communication, first using GSM but now via mobile broadband including G3
and EDGE technology. Where mobile communications are not available, alternative solutions
including meteor burst communication are available giving a low cost alternative to satellite
data transmission.

The development of EM transmission solutions for sensor installations allows the installation
of sensors without cables, while inductive power transmission can simplify the installation of
sensors in boreholes or difficult to access locations.

Fiber optic sensors provide the opportunity for simple deployment of vast sensor arrays for
the measurement of various parameters, such as strain, temperature and pressure. Multiple
sensor points can be included in a single optical fiber cable. NGI has deployed such a system
including thousands of strain and temperature measurements for a bridge monitoring system
in Sweden.

The implementation of database data collection combined with on-demand web pages and
well-designed graphical interfaces are vastly improving data accessibility and interpretation.
This approach provides data users with the ability to browse data and generate information as
required.

This well-filled engineering toolbox is aiding the development of Early Warning Systems,
where the application of technology is used to reduce risk and improve safety for society.

REFERENCES
Conflicts Working Group Publication P00-302, Social Science Research Centre,
Berlin.
photogrammetry techniques to monitor landslide bodies.” XXth ISPRS Congress, Geo-
Imagery Bridging Continents, Istanbul.
MESPOSH Consortium, confidential project report.
United States Environmental Protection Agency (EPA) (2005). Technologies and Techniques
for Early Warning Systems to Monitor and Evaluate Drinking Water Quality: A State-
of-the-Art Review. Office of Research and Development, Report No. EPA/600/R-
05/156.
Kääb, A., Huggel, C., Fischer, L., Guex, S., Paul, F., Roer, I., Salzmann, I., Schlæfli, S.,
of glacier- and permafrost-related hazards in highmountains: an overview.” Natural
Hazards and Earth System Sciences, 5, 527–554.


DEVELOPMENT OF A METHOD FOR MULTI-SCALE LANDSLIDE RISK ASSESSMENT IN CUBA

C. J. van Westen
International Institute for Geoinformation Science and Earth Observation (ITC)
The Netherlands

E. A. Castellanos Abella
Instituto de Geología y Paleontología (IGP), Cuba

Abstract: This paper presents a summary of the method and the results of landslide risk assessment carried out in Cuba as a contribution to the system of multi-hazard risk assessment by the Civil Defence authorities. The method is developed at four different scales, national, provincial, municipal and local, each with specific objectives. At the national level a landslide risk index was generated, using a semi-quantitative model with 10 indicator maps using spatial multi-criteria evaluation techniques in a GIS system. Each indicator was standardized according to its contribution to hazard and vulnerability. The indicators were weighted using direct, pairwise comparison and rank ordering weighting methods and weights were combined to obtain the final landslide risk index map. The results were analysed per physiographic region and administrative units at provincial and municipal levels. The hazard assessment at the provincial scale follows a method for combined heuristic and statistical landslide susceptibility assessment, its conversion into hazard, and the combination with elements at risk data for vulnerability and risk assessment. The method is tested in Guantánamo province. For the susceptibility analysis, 12 factors maps were considered: geomorphology, geology, soil, landuse, slope, aspect, internal relief, drainage density, road distance, fault distance, maximum daily rainfall and peak ground acceleration. Five different landslide types were analyzed separately (small slides, debris flows, rockfalls, large rockslides and topples). The susceptibility maps were converted into hazard maps, using the event probability, spatial probability and temporal probability. Semi-quantitative risk assessment was done by applying the risk equation in which the hazard probability is multiplied with the number of exposed elements at risk and their vulnerabilities. At the municipal scale, a detailed geomorphological mapping formed the basis of the landslide susceptibility assessment. A heuristic model was applied to a municipality of San Antonio del Sur in Eastern Cuba. The study is based on a terrain mapping units (TMU) map, generated at 1:50,000 scale by interpretation of aerial photos and satellite images and field data. Information describing 603 terrain units was collected in a database. Landslide areas were mapped in greater detail to classify the different failure types and parts. The different landforms and the causative factors for landslides were analyzed and used to develop the heuristic model. The model is based on weights assigned by expert judgment and organized in a number of components such as slope angle, internal relief, slope shape, geological formation, active faults, distance to drainage, distance to springs, geomorphological subunits and existing landslide zones. At the local level, digital photogrammetry and geophysical surveys were used to characterize the volume and failure mechanism of the Jagüeyes landslide. In order to improve the temporal probability information for Cuba, the generation of a national landslide inventory database is essential.
INTRODUCTION
Due to natural conditions or man-made actions, landslides have produced considerable human and economic losses (Schuster & Fleming 1986; Guzzetti 2000). Individual slope failures are generally not so spectacular or so costly as earthquakes, major floods, hurricanes or some other natural catastrophes. However, they are more widespread, and over the years they may cause more damage to properties than any other geological hazards (Varnes & IAEG-Commission on Landslides and Others Mass Movements on Slopes 1984). In Cuba, most of the studied landslides are associated with hurricanes, tropical storms or prolonged periods of rainfall (Viña et al. 1977; Formell & Albear 1979; Díaz et al. 1983; Pérez 1983; Iturralde-Vinent 1991; Magaz et al. 1991; Castellanos et al. 1998b; Pacheco & Concepción 1998; Castellanos 2000). Since the landslide damage is recorded as associated to the main disaster, there is no information on how many landslides have happened and where they are located. From 1785 up to 1984, a total of 108 hurricanes have passed over Cuba, of which 23 were of high intensity (>200 km/h), 38 of moderate intensity (151-200 km/h) and 47 of low intensity (118-150 km/h) (Rodríguez 1989). So far there are no official records for landslides related to hurricane events. All disasters damages were included in the hurricane data and no detailed description was made for the secondary disasters like flooding or landslides. Even though, in a report presented by the National Civil Defence Headquarters, it was recognized that 45,000 inhabitants are vulnerable to landslides (EMNDC 2002). Due to the lack of a landslide inventory, the knowledge about geological, geomorphological, tectonic and hydrological conditions under which these events could happen is limited in Cuba.

This research is intended to contribute in reducing the lack of knowledge about landslide problems mentioned by Cuba as well as in applying innovative spatial analysis for landslide risk assessment at different scales, taking into account the specific situation with respect to data availability and landslide types in Cuba. Various methods and models for landslide hazard and risk assessment have been applied in other countries, but they need to be translated to the Cuban situation. This research is dealing with multi-scale landslide risk assessment in Cuba. The main objective is to design a framework for spatial landslide risk assessment in Cuba, considering a multi-level approach and the specific characteristics of Cuba related to landslide types and distribution, availability and organizational structure. To do so, a set-up of a national landslide inventory database was made and landslide risk assessment methodology was worked out for four administrative levels and study areas, each one with a different scale, objectives, available datasets and analysis techniques.

This paper gives a summary of the methods used and main findings of landslide risk assessment of 4 different scales. More detailed descriptions can be found in Castellanos & Van Westen (2001a, 2001b, 2005, 2007a, 2007b, 2008). Before presenting the case study results, the general framework for spatial landslide risk assessment in Cuba and the location of the case study areas are presented.

CUBAN CIVIL DEFENCE SYSTEM AND MULTI-HAZARD RISK ASSESSMENT
Cuba is considered a model in hurricane risk management by the United Nations (Sims & Vogelmann 2002; ISDR 2004) because hurricanes in Cuba cause considerable less casualties as compared to neighboring countries with a different economical, social and political context such as Haiti, Jamaica or even compared with the USA (Wisner et al. 2006). The reasons for this relate to “an impressive multi-dimensional process” using as foundation of “a socio-economic model that reduces vulnerability and invests in social capital through universal access to government services and promotion of social equity” (Thompson & Gaviria 2004).
The disaster reduction in Cuba is controlled by the Civil Defence, an organization which has its roots in the revolution of 1959. The National Revolutionary Militia (MNR in Spanish) was created, and in 1961 the Military Organization of Industries (OMI in Spanish), and its main duty was vigilance and protection of economic and political targets in the Country. In 1962, OMI transformed into the Central Headquarter of Popular Defense, commonly known as Popular Defense, which was organized in all different levels (provinces, regions, municipalities, etc.) and later renamed into Civil Defence. The first main test with respect to natural disasters was in October 1963, when the country was severely affected by hurricane Flora, causing about 1200 casualties. This event changed the responsibilities of the Cuban Civil Defence and natural disasters response was added. The tasks of the Civil Defence at that time included the organization of a warning system, emergency response planning and to plan how to continue production during military aggressions and natural disasters. Since 1986 an annual disaster response simulation exercise, called “Meteoro”, is conducted on an annual basis. Initially the exercise was designed to be better prepared for cyclone season (June-November), but gradually started to include all other disaster types in all disaster management levels with high involvement of local population.

In the past decades, the Civil Defence had to deal with numerous natural, technological and sanitary disasters which lead to a substantial improvement of the organization. Since then, the territories and local authorities started to have a broader view of disasters considering all different scenarios including dam breaks, chemical contamination, epidemics, etc. Also authorities and Civil Defence started to pay more attention to prevention measures besides the original focus on response. In 1997, the structural organization of the Civil Defence was established as shown in Figure 1.

![Figure 1: Structural organization of Civil Defence System in Cuba and risk assessment (after EMNDC 2007)](image)

The role of the Civil Defence was re-established with several functions. The first one was to identify and evaluate, in coordination with the organizations, enterprises and social institutions, the hazard, vulnerability and risk factors as well as to provide the planning needed to cope with them. Many laws have articles related to natural disasters reduction. In 2004, Cuba was hit by two major hurricanes in a relative short period: Ivan and Charley.
After this it was decided that each territory should have a disaster reduction plan and disasters reduction measures will be included in the social-economic plan every year. The importance of the Civil Defence system in Cuba is illustrated in Table 1, which gives some statistics related to the major hurricanes that have affected Cuba.

Table 1: Statistics of disaster management in Cuba for 11 selected storms. DT: Tropical depression. (Source: National Civil Defence)

<table>
<thead>
<tr>
<th>Cyclone</th>
<th>Year</th>
<th>Evacuated</th>
<th>In shelter</th>
<th>Transport</th>
<th>Mobilized</th>
<th>Deaths</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kate</td>
<td>1985</td>
<td>473,400</td>
<td>143,200</td>
<td>14,600</td>
<td>41,800</td>
<td>2</td>
</tr>
<tr>
<td>Lili</td>
<td>1996</td>
<td>421,200</td>
<td>276,700</td>
<td>5,600</td>
<td>74,500</td>
<td>0</td>
</tr>
<tr>
<td>Georges</td>
<td>1998</td>
<td>818,800</td>
<td>215,200</td>
<td>10,300</td>
<td>118,100</td>
<td>6</td>
</tr>
<tr>
<td>Mitch</td>
<td>1998</td>
<td>50,600</td>
<td>1,900</td>
<td>1,800</td>
<td>22,400</td>
<td>0</td>
</tr>
<tr>
<td>Irene</td>
<td>1999</td>
<td>33,600</td>
<td>11,200</td>
<td>1,500</td>
<td>12,600</td>
<td>4</td>
</tr>
<tr>
<td>Michelle</td>
<td>2001</td>
<td>783,400</td>
<td>166,300</td>
<td>6,100</td>
<td>102,400</td>
<td>5</td>
</tr>
<tr>
<td>Isidore</td>
<td>2002</td>
<td>307,000</td>
<td>34,500</td>
<td>2,700</td>
<td>48,800</td>
<td>0</td>
</tr>
<tr>
<td>Lili</td>
<td>2002</td>
<td>385,300</td>
<td>56,300</td>
<td>5,000</td>
<td>81,700</td>
<td>1</td>
</tr>
<tr>
<td>DT No. 14</td>
<td>2002</td>
<td>70,000</td>
<td>3,300</td>
<td>1,500</td>
<td>20,900</td>
<td>0</td>
</tr>
<tr>
<td>Charley</td>
<td>2004</td>
<td>224,449</td>
<td>35,749</td>
<td>2,444</td>
<td>45,082</td>
<td>4</td>
</tr>
<tr>
<td>Iván</td>
<td>2004</td>
<td>2,226,066</td>
<td>416,123</td>
<td>13,016</td>
<td>215,122</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>5,793,815</td>
<td>1,360,472</td>
<td>64,560</td>
<td>783,404</td>
<td>22</td>
</tr>
</tbody>
</table>

The official responsibility as main coordinator for conducting risk assessment in every municipality (169 in all) was assigned to the Ministry of Science, Technology and Environment (CITMA in Spanish) as shown in Figure 1. Many other organizations are involved depending on the hazard type, making a multidisciplinary team for risk assessment. The minimum spatial unit was set up at the Popular Council or Defense Zone, a spatial administrative unit lower than a municipality. The main idea was to establish a methodology where risk could be compared spatially (among municipalities) and temporally (during years) in order to provide priorities and to monitor the progress in risk reduction. The management of risk reduction is seen as an obligation of the State which includes all organizations involved. For its implementation every municipality and province should have a Management Centre for Risk Reduction (Figure 1) with financial support from UNDP. These centres have the following main functions i) periodic assessment, evaluation and monitoring of the risk in the territory, ii) support with equipment and information the Council of Defense (municipal and provincial) during respond and recovering phases, iii) record actions taken in disaster reduction and iv) contribute in training local people as well as dissemination of measures for disaster reduction (EMNDC 2002). These centres should also collect historic data about previous disasters besides receiving periodic information from the different early warning systems. In 2006, CITMA started the multi-hazard risk assessment with the 15 municipalities of the province of Havana City. Initially, events were mainly associated with tropical cyclones or intense rainfalls which included: flooding, sea surge and strong winds. The general procedure has 4 stages (AMA 2007) as indicated in Figure 2.
Figure 2: Stages for multi-hazard risk assessment in Cuba (after AMA 2007) and main tasks for landslide risk assessment. Terminology used as in the reference

Spatial Landslide Risk Assessment

Figure 3 presents the proposed methodological framework for spatial landslide risk assessment which was used in the different scales, with specific adaptations according to the scale, objectives and available data. Four major steps have been identified. Data collection for landslide risk assessment is the starting point where frequency analysis could be carried out to landslide inventory, rainfall or earthquake databases. This part usually consumes most of the time in a landslide risk assessment project. The landslide susceptibility and hazard assessment is the best known part for landslide studies. Landslide vulnerability assessment is probably the weakest part in the whole process since relatively little work has been done on the quantification of physical vulnerability due to landslides (Van Westen et al. 2005).

Figure 3: Methodological framework for spatial landslide risk assessment (Castellanos 2008)
CASE STUDY AREAS

The multi-scale assessment methods were applied at national, provincial, municipal and local levels (See Figure 4). At national level, the whole country was analysed for landslide risk. At this level, the main objective was an initial screening process to recognize main administrative areas of national interest for landslides studies. The data used came mainly from national data providers and the DEM derivatives were obtained with SRTM data. The method and results are mainly qualitative by implementing spatial multi-criteria evaluation techniques. National risk index map was compared with provincial and administrative units for ranking priorities in landslide research policies. As this level of analysis is very general, only broad considerations are taken into account and the results should be considered as “indicative”.

For the provincial assessment, the Guantánamo province was selected with half a million population and very diverse natural environments. In this case study, a more semi-quantitative approach was carried out supported by a landslide inventory. Landslides were divided by five main types of movements and also five types of elements at risk were considered. Artificial neural network, weight of evidence and spatial multi-criteria evaluation methods were applied. The analysis explored qualitative and semi-quantitative approach for producing different results with the objectives of locate high risk areas, identify main causes and alert provincial and municipal authorities for further actions.

At municipal level San Antonio del Sur, a municipality located inside Guantánamo, was selected (see Figure 4). The objective at this level was to identify specific hazard probabilities and landslide risk associated to delineate areas for disaster risk reduction plans. In this scale, a more detailed photo-interpretation identified individual features as well as historical events that could be classified. Heuristic approach supported by many fieldwork campaigns allowed applying multi-criteria evaluation techniques using semi-quantitative and quantitative approaches depending on the element at risk. The results were integrated into the municipal disaster reduction plan. Inside this municipality, there is long escarpment called Sierra de Caujerí which was the study area at local level. This level actually includes two areas at two scales, the escarpment and the Jagüeyes landslide located in the centre of the escarpment. Analysis carried out in both areas complement each other by making geophysical survey and photogrammetrical analysis. The reconstruction of Jagüeyes disaster back in 1963 and a detailed survey of elements at risk in the escarpment allowed a more quantitative landslide risk assessment establishing risk buffer zones.
Figure 4: Case study areas for multi-scales landslide risk assessment (Castellanos 2008)

SPATIAL DATA
Although Cuba is a developing country, the context regarding information for landslide risk assessment could be different than in other developing countries. In Cuba, the situation may be less problematic concerning the existence of the data but more difficulties are present in access and format of the data. Landslide risk assessment requires an extensive and multi-disciplinary dataset (Van Westen et al. 2008). During this research, many dataset were collected from diverse organizations.

Table 2 shows the main data sets for landslide risk assessment and the providers for this data in Cuba at different levels used during this research. At national level, most spatial information was collected from the national atlas of the country and from national organizations including the statistics data from the national statistic office. Data collection for the provincial assessment was one of the most extensive. Here, as in other more detailed areas, the data about landslides was collected mainly by extensive photo interpretation and fieldwork campaigns. Elevation data of good resolution was processed to obtain geomorphometric maps. For municipal landslide risk assessment at 1:50,000 scale, most data, including the elements at risk, were digitized from topographic maps.
Table 2: Data providers or source for landslide risk assessment in Cuba (Castellanos 2008)

<table>
<thead>
<tr>
<th>Data layer and types</th>
<th>Levels of analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>National 1:1,000,000</td>
</tr>
<tr>
<td>Landslides inventory</td>
<td>-</td>
</tr>
<tr>
<td>Terrain mapping units</td>
<td>-</td>
</tr>
<tr>
<td>Geomorphology</td>
<td>Atlas</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Digital Elevation</td>
<td>SRTM</td>
</tr>
<tr>
<td>Model (DEM)</td>
<td></td>
</tr>
<tr>
<td>Slope angle</td>
<td>From SRTM</td>
</tr>
<tr>
<td>Slope orientation</td>
<td>-</td>
</tr>
<tr>
<td>Slope shape</td>
<td>-</td>
</tr>
<tr>
<td>Internal relief</td>
<td>-</td>
</tr>
<tr>
<td>Drainage (density)</td>
<td>-</td>
</tr>
<tr>
<td>Springs</td>
<td>-</td>
</tr>
<tr>
<td>Geology</td>
<td>Atlas</td>
</tr>
<tr>
<td>Soils</td>
<td>-</td>
</tr>
<tr>
<td>Faults</td>
<td>Atlas</td>
</tr>
<tr>
<td>Landuse</td>
<td>Atlas</td>
</tr>
<tr>
<td>Water table</td>
<td>-</td>
</tr>
<tr>
<td>Rainfall and maximum</td>
<td>ISMET</td>
</tr>
<tr>
<td>probabilities</td>
<td></td>
</tr>
<tr>
<td>Earthquakes and seismic</td>
<td>CENAIS</td>
</tr>
<tr>
<td>acceleration</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>ONE</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Roads</td>
<td>Atlas</td>
</tr>
<tr>
<td>Lifeline utility systems</td>
<td>-</td>
</tr>
<tr>
<td>Housing</td>
<td>INV</td>
</tr>
<tr>
<td>Building</td>
<td>-</td>
</tr>
<tr>
<td>Production</td>
<td>ONE</td>
</tr>
<tr>
<td>Facilities</td>
<td>-</td>
</tr>
<tr>
<td>Protected areas</td>
<td>UICN</td>
</tr>
</tbody>
</table>

NATIONAL SCALE ANALYSIS

Only limited research has been done on landslide risk assessment for large areas or entire countries (Guzzetti 2000; Yoshimatsu & Abe 2006). At such small scales, the aim is to produce a landslide risk index which makes it possible to zoom in on the high risk areas for more detailed studies. Risk indexes have been applied in small scale studies either for specific countries (Carreño et al. 2007) or at a global level (Evans & Roberts 2006; Nadim et al. 2006a, b). For designing the vulnerability indicators, it is necessary to take into account the socio-economic conditions, which may vary from country to country. In general vulnerability can be divided in four different types, such as physical, social, economic and environmental which can be combined to derive a qualitative index. There are relatively few publications related to landslide vulnerability assessment and most of them are dealing with large scale studies or on a site-investigation scale (Glade et al. 2005). On a very small scale
such as a national landslide risk assessment, it is not feasible to represent the degree of impact depending on the magnitude of the hazardous event and the characteristics of the elements at risk.

Unfortunately there is no national landslide inventory available for Cuba. Recently a project has started to develop such a database. The current national landslide database only contains those landslides where major damage has been reported and is therefore not complete both in space and time. Also quantitative damage information is not available for most of the landslides in the database. For that reason in this study the national landslide database was used with caution as it does not give a complete picture for the country yet. If a complete landslide database would have been available, it could have served as the main input in the landslide risk index. The density of landslides per municipality could then have been used as the main hazard indicators, and the landslide damage per municipality as the main vulnerability indicator. As part of the national landslide risk assessment project for the National Civil Defence, a research project was also initiated to improve the national landslide inventory, making use of local Civil Defence personnel that are trained in reporting the occurrence of new landslides, combined with multi-temporal landslide maps based on Remote Sensing (Castellanos & Van Westen 2005).

Considering the objectives for the assessment of a national landslide risk index map in combination with a large study area and limitations in available data, a semi-quantitative approach was selected. The semi-quantitative estimation for landslide risk assessment is considered useful in the following situations: as an initial screening process to identify hazards and risks, when the level of risk (presumably) does not justify the time and effort or where the possibility of obtaining numerical data is limited (Australian Geomechanics Society and Sub-committee on Landslide Risk Management 2000). Semi-quantitative approaches consider explicitly a number of factors influencing the stability. A range of scores and settings for each factor may be used to assess the extent to which that factor is favourable or unfavourable to the occurrence of instability (hazard) and the occurrence of loss or damage (consequence).

The landslide risk was represented at this scale by a semi-quantitative risk index. For implementing the semi-quantitative model, the spatial multi-criteria evaluation (SMCE) module of ILWIS GIS was used. SMCE application assists and guides users in doing multi-criteria evaluation in a spatial manner (ITC 2001). The input is a set of maps which are the spatial representation of the criteria. They are grouped, standardized and weighted in a “criteria tree”. The output is one or more “composite index map(s)”, which indicates the realization of the model implemented. The theoretical background for the multi-criteria evaluation is based on the analytical hierarchical process (AHP) developed by Saaty (1980).

As mentioned earlier, the quantification of the expected losses for landslides is not possible, given the limitations in data availability and size of the study area. A schematic representation of the landslide risk assessment model is given in Figure 5. The landslide risk index is high only if both the hazard and vulnerability index maps are high. The hazard component in fact only represents landslide susceptibility, as it doesn’t include the time factor required for estimating probability, because of lack of sufficient temporal landslide information for the country. The intermediate map of hazards is constructed again by multiplying two other intermediate maps of Conditions and Triggering Factors. Conditions are the intrinsic environmental parameters of the terrain that lead to particular susceptibility for landslide occurrence and Triggering Factors are the most frequent triggering mechanisms
that make landslide event happen. The intermediate map of Vulnerability is generated by combining the four vulnerability types mentioned earlier.

Figure 5: Landslide risk assessment model at national level in Cuba (Castellanos & van Westen 2007a)

After the selection of the indicators, their standardization and the definition of indicator weights, the analysis was carried out using an ILWIS GIS script to obtain the composite index maps and the final landslide risk index map (Figure 6). The frequency of the risk index values is highly influenced by the large number of pixels with zero values, which were introduced using a mask for flat areas. Without considering zeros, the risk index values range from 0.022 to 0.620 with a mean of 0.18, a median of 0.170 and a predominant value of 0.097. These values are low due to the multiplication of the intermediate maps of Hazard and Vulnerability, which were made using the weights as shown in Table 3.

The landslide risk index map shows the spatial distribution of the relative risk values for the entire country. It is possible to recognize the areas with higher values and to query the database of indicator maps to search the causes of these higher values as a backward analysis. Due to the characteristics of the available data sets, it is not possible to avoid polygon boundaries especially with the vulnerability indicators related to administrative units, the geological units and the land use types. For a more detailed study, the risk index values were analysed physiographically and administratively at provincial and municipal level.

The resulting landslide risk index is not a static one, as a number of indicators have a temporal variability, and the landslide risk index map should therefore be updated regularly. Similarly, the model equation could be improved by adding new indicators, once more data becomes available, and by fine-tuning the standardization and weights values. Depending on further requests from the end-user, the model can also be made more complex, and made at a higher spatial resolution. The use of landslide risk index statistics for provinces and
municipalities is useful for ranking them in order of importance for landslide risk reduction measures.

Table 3: Overview of indicators (italic), intermediate maps or sub-goals (bold), with their corresponding weight values. The weighting and standardization method is indicated in the right columns (Castellanos & van Westen 2007a)

<table>
<thead>
<tr>
<th>National Landslide Risk Model</th>
<th>Weighting</th>
<th>Standardization</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hazard</td>
<td>Direct</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Direct</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Concave</td>
<td>Ranking</td>
</tr>
<tr>
<td></td>
<td>Ranking</td>
<td></td>
</tr>
<tr>
<td>Conditions</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.8</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>0.20</td>
<td>0.20</td>
<td></td>
</tr>
<tr>
<td>0.30</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td></td>
<td>0.90</td>
<td>Maximum</td>
</tr>
<tr>
<td></td>
<td>0.10</td>
<td>Maximum</td>
</tr>
<tr>
<td>Factors</td>
<td>Pair-wise</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constraint for hazard map, areas with slope angle 3 degrees or less.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>Ranking</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Concave</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
</tr>
<tr>
<td>0.256667 Housing</td>
<td>Rank</td>
<td></td>
</tr>
<tr>
<td>0.090000 Transportation</td>
<td>Expected</td>
<td></td>
</tr>
<tr>
<td>0.456676 Population</td>
<td>Value</td>
<td></td>
</tr>
<tr>
<td>0.156667 Production</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.040000 Protected areas</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4: Percentage of each province with low, moderate and high landslide risk (Castellanos & van Westen 2007a)

<table>
<thead>
<tr>
<th>Province</th>
<th>Low risk %</th>
<th>Moderate risk %</th>
<th>High risk %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinar del Río</td>
<td>74.8</td>
<td>24.1</td>
<td>1.2</td>
</tr>
<tr>
<td>La Habana</td>
<td>88.6</td>
<td>8.4</td>
<td>3.1</td>
</tr>
<tr>
<td>Ciudad de La Habana</td>
<td>84.8</td>
<td>4.5</td>
<td>10.7</td>
</tr>
<tr>
<td>Isla de la Juventud</td>
<td>97.4</td>
<td>2.6</td>
<td>0.0</td>
</tr>
<tr>
<td>Matanzas</td>
<td>96.5</td>
<td>3.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Cienfuegos</td>
<td>82.0</td>
<td>17.8</td>
<td>0.3</td>
</tr>
<tr>
<td>Villa Clara</td>
<td>86.7</td>
<td>8.6</td>
<td>4.8</td>
</tr>
<tr>
<td>Sancti Spiritus</td>
<td>79.1</td>
<td>20.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Ciego de Avila</td>
<td>96.5</td>
<td>3.2</td>
<td>0.3</td>
</tr>
<tr>
<td>Camaguey</td>
<td>96.9</td>
<td>2.9</td>
<td>0.2</td>
</tr>
<tr>
<td>Las Tunas</td>
<td>98.5</td>
<td>1.4</td>
<td>0.1</td>
</tr>
<tr>
<td>Holguin</td>
<td>64.3</td>
<td>9.6</td>
<td>26.1</td>
</tr>
<tr>
<td>Granma</td>
<td>74.7</td>
<td>3.6</td>
<td>21.8</td>
</tr>
<tr>
<td>Santiago de Cuba</td>
<td>35.9</td>
<td>14.3</td>
<td>49.8</td>
</tr>
<tr>
<td>Guantánamo</td>
<td>31.6</td>
<td>37.2</td>
<td>31.2</td>
</tr>
</tbody>
</table>

The method allows evaluating which of the indicators is responsible for high risk index values. Local (provincial and municipal) authorities can now be warned about the landslide risk that their areas are facing and since they are part of the civil defence system in Cuba, they can also allocate resources for a local landslide mitigation program. The city of Santiago
de Cuba ranks at the top of the landslide risk index list of municipalities as this densely populated area is located along the Sierra Maestra mountainous system.

The results are intended to support national decision makers in prioritizing funding for risk assessments at local, municipal and provincial levels. With the outcomes of the study in Cuba, the Civil Defence organisation will be able to alert local authorities about the risk levels and to link the information to the national hurricane early warning system, allowing also warning and evacuation for landslides prone areas.

PROVINCIAL SCALE ANALYSIS

The method for landslide risk assessment at a provincial scale was developed and tested in the province of Guantánamo (see Figure 4), the most eastern province of Cuba. Guantánamo contains both the most humid (in the North) and driest (in the South) zones of Cuba. The province has 10 municipalities and 386 settlements from where 18 are considered urban. Agriculture is the most important economic income for the province which is based on sugar cane, coffee, cacao, wood and coconut. The last four are cultivated in mountainous regions. The industries include an iron foundry, and factories for coffee, agricultural tools, furniture, food, sugar cane and salt. Guantánamo has the record of 49 devastating hurricanes measured over the period 1789-2003, which are more frequent in September and October. Since 1997 to 2002, there were 93 forest fires reported, affecting an area of 3043 hectares. The landslides resulting from these other disasters are rarely recorded in the official statistics. The province also has a substantial earthquake hazard, due to the presence of the Caribbean-North American plate boundary in the south.

A schematic overview of the methodology is given in Figure 7. The method started with a comprehensive landslide inventory, and the collection of input data on landslide causal factors and elements at risk, represented in the upper part of Figure 7. Landslides were photo-interpreted from 300 aerial photos (format 23 x 23mm) from the year 2000 at 1:25,000 scale, covering the entire Guantánamo province. Unfortunately no multi-temporal image interpretation could be carried out, which made it difficult to establish the age of the landslides. In total 281 landslides were identified covering an area of about 19.92 km². From this inventory, five main types of landslides were determined: rockfalls, debris flows, topples, small landslides and large rockslides. The next step was to generate a number of landslide susceptibility maps for the five different landslide types, using a combination of a heuristic approach, and of several statistical methods, such as the Weights of Evidence Modeling and Artificial Neural Network analysis. The susceptibility maps were converted into hazard maps, based on the landslide densities of the susceptibility classes and the temporal probability of landslide occurrence. This resulted in five hazard maps (H_slide to H_rockslide, indicated in the middle part of Figure 7.

Elements at risk (EaR) data were collected for population, roads, essential facilities and non residential buildings, agricultural land use, and protected areas. In order to estimate the risk to these elements by the five different landslide types, each of the five hazard maps was overlain with the elements at risk maps to calculate the number of elements per hazard class. In the lower part, the method for the risk assessment is presented. The study was based on raster analyses using ILWIS and ArcGIS© GIS software at 50m resolution taking into account the cartographic rule of a maximum detail of 0.5 mm at the scale of the final map (1:100,000 scale in this case), resulting in maps with 2475543 pixels.
Figure 6: Final landslide risk index map, as presented to the National Civil Defence authorities. Inset map in the upper right corner indicates the landslide risk index per municipality, both as the percentage of area with landslide risk index larger than 0 (coverage) as well as the average landslide risk index. The bar chart shows the landslide risk index values per province (Castellanos & van Westen 2007a).
The casual factor maps were selected based on literature and on the data available in Cuba. They were separated into 3 groups: morphometric factors, ground conditions, distance related factors, and triggering factors. A DEM was created using the ArcGIS® “topo to raster” tool, and four morphometric parameter maps were extracted: slope steepness, slope orientation (aspect), internal relief (vertical dissection) and drainage density. Existing geological, geomorphological and soil maps were used and reclassified by reducing the number of legend units to only those that were considered relevant for landslide susceptibility assessment. The landuse map, which was also obtained from existing maps, was used both as potential causal factor, and as element at risk for estimating the impact of landslides on agricultural production. Two buffer maps were used: distance to main roads in sloping areas, and distance to active faults. Also two triggering factors were used in the landslide hazard assessment. The first of these was a raster map of maximum expected rainfall in 24 hours for a 100 year return period, and the was a map of the peak ground acceleration (PGA) with a 10 percent exceedance probability in 50 years.

As part of the hazard analysis, two methods were applied for estimating spatial probabilities: Weights of Evidence (WoE) modeling (Bonham-Carter 1996) and Artificial Neural Network (ANN) analysis (e.g. Lee et al. 2004). The selection of the relevant causal factor maps for each of the five landslide types was made based on initial results of WoE modeling, and expert judgment. The susceptibility maps for 4 out of 5 landslide types were generated using WoE modeling, as for each of these, the main casual factors could be clearly separated, and also because the number of events for each of these was relatively small. For the generation of the susceptibility map of slide-type movements, it was decided to use the ANN method, because there were several different causal mechanisms for this landslide type, that were difficult to separate, and also because the number of events was substantially larger than for the other types. The landslide inventory database was randomly subdivided in three subsets: a
training set (75% of the landslides) used to optimize the weights, a validation set (12.5%) used to stop the network algorithm before the network starts learning from noise in the data, and a test set (12.5%) to evaluate the prediction capability of the network. An equal number of samples was also randomly taken in non-landslide areas. Due to the small scale of the study and the relatively large pixel size it was decided not to include a runout analysis as part of the landslide susceptibility assessment. The five susceptibility maps were validated using the landslide inventory and success rate curves were generated (See Figure 8). The results showed generally a very good fit, especially for the topples, large rockslides and rockfalls that occur under very specific conditions. The success rate curves were also used to classify the susceptibility maps with approximately equal percentages of the total number of landslides (e.g. ~ 70 % of all landslides in the highest class). The susceptibility maps are shown in Figure 9. For the spatial landslide vulnerability analysis, only five types of elements at risk (population, facilities, roads, protected areas and landuse) were used. Also here the approach was to combine them using spatial multi-criteria evaluation (SMCE). As explained before, since it was not possible to obtain monetary values for all elements at risk, the vulnerability was carried out using relative weight values.

The risk assessment was carried out for the 5 different landslide types and 5 types of elements at risk, using both a qualitative and semi-quantitative method.

![Figure 8: Success rate curves for the five landslide susceptibility maps](image-url)
Figure 9: Landslide susceptibility maps. A: debris flows, B: large rockslides, C: rockfalls, D: topples, E: slides and F: all hazards (Castellanos 2008)

**Qualitative Risk Assessment**
The qualitative landslide risk assessment initiated by overlaying each of the four hazard maps (for each landslide type) with a composite vulnerability map. The map resulting from the overlaying was reclassified considering the hazard and vulnerability classes. In order to be more conservative with the vulnerability assessment, an “expert-based” approach was applied to create the qualitative landslide risk map. The rules used for the combination of hazard and vulnerability classes are given in Table 5. With this approach, the areas with low or even no vulnerability but low hazard, were still considered as risk areas (23.48%). This classification was adopted taken into account the subjectivity and the data problems involved in the analysis.
Table 5: Qualitative landslide risk matrix applied in Guantánamo province. The percentage of the total area for each combination is given in brackets (Castellanos 2008)

<table>
<thead>
<tr>
<th>Vulnerability</th>
<th>High</th>
<th>Moderate</th>
<th>Low</th>
<th>Not</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>No risk (0.05)</td>
<td>High risk (0.00)</td>
<td>High risk (0.00)</td>
<td>High risk (0.00)</td>
</tr>
<tr>
<td>Moderate</td>
<td>No risk (0.12)</td>
<td>Moderate risk (0.02)</td>
<td>Moderate risk (0.01)</td>
<td>High risk (0.00)</td>
</tr>
<tr>
<td>Low</td>
<td>No risk (3.67)</td>
<td>Low risk (0.48)</td>
<td>Moderate risk (0.17)</td>
<td>High risk (0.06)</td>
</tr>
<tr>
<td>Not</td>
<td>No risk (41.86)</td>
<td>Low risk (23.48)</td>
<td>Low risk (17.27)</td>
<td>Moderate risk (12.81)</td>
</tr>
</tbody>
</table>

Hazard

Figure 10: Qualitative landslide risk map of Guantánamo province (Castellanos 2008)

Semi-Quantitative Analysis
The first step in the semi-quantitative risk analysis was the conversion of the susceptibility maps into hazard maps. For this, three probabilities were calculated for pixels belonging to each hazard class within the five maps:

- Event probability, \( P(E) \), defined as the probability that if a landslide occurs of a given type, it happens in the particular susceptibility class.
- Spatial probability, \( P(S) \), defined as the probability that if a landslide occurs within a given susceptibility class, a pixel in this class might be hit.
- Temporal probability, \( P(T) \), defined as the annual probability of occurrence of a particular landslide type.
The event probability and spatial probability were calculated based on the area of landslides within each susceptibility class, in relation to either the total area of landslides (for P(E)) or the total area of the class (for P(S)). Temporal probability was the most difficult to estimate, also in the absence of a historical landslide database. Therefore, based on geomorphological analysis and comparison with return periods for the main triggering events, a return period (RP) of 100 years was selected for large rockslides, a 50 years RP for rockfall and topples, and a 20 year RP for debris flows and slides.

The semi-quantitative analysis of the expected number of people that might be killed by landslides annually in the province was done by overlaying the hazard maps with a population distribution map, indicating the maximum number of persons in buildings per pixel of 50 by 50 m. Outdoor population and temporal variations of population density were not considered. This results in the number of persons per hazard class as indicated in Table 6. The next step was to estimate the vulnerability of people being killed by a landslide while being indoors, based on the type of landslide and the expected magnitude of the event. These values were based on literature (e.g. Glade et al. 2005) and expert judgment, in the absence of historical landslide damage information.

<table>
<thead>
<tr>
<th></th>
<th>Low hazard</th>
<th>Moderate hazard</th>
<th>High hazard</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Rockfall</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazard</td>
<td>5.30E-07</td>
<td>1.71E-05</td>
<td>7.52E-04</td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>1616</td>
<td>522</td>
<td>200</td>
<td>2338</td>
</tr>
<tr>
<td>Specific risk</td>
<td>0.0005</td>
<td>0.0054</td>
<td>0.0902</td>
<td>0.0961</td>
</tr>
<tr>
<td><strong>Rockslides</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazard</td>
<td>2.69E-06</td>
<td>5.36E-05</td>
<td>1.67E-03</td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>1265</td>
<td>453</td>
<td>184</td>
<td>1902</td>
</tr>
<tr>
<td>Specific risk</td>
<td>0.0034</td>
<td>0.0243</td>
<td>0.3072</td>
<td>0.3349</td>
</tr>
<tr>
<td><strong>Topples</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazard</td>
<td>1.17E-06</td>
<td>1.36E-05</td>
<td>4.48E-04</td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>2139</td>
<td>745</td>
<td>12</td>
<td>2896</td>
</tr>
<tr>
<td>Specific risk</td>
<td>0.0005</td>
<td>0.0020</td>
<td>0.0011</td>
<td>0.0036</td>
</tr>
<tr>
<td><strong>Debris flows</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazard</td>
<td>1.27E-07</td>
<td>7.40E-06</td>
<td>9.48E-05</td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>8047</td>
<td>553</td>
<td>0</td>
<td>8600</td>
</tr>
<tr>
<td>Specific risk</td>
<td>0.0004</td>
<td>0.0016</td>
<td>0.0000</td>
<td>0.0020</td>
</tr>
<tr>
<td><strong>Slides</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hazard</td>
<td>6.24E-07</td>
<td>2.60E-05</td>
<td>3.06E-04</td>
<td></td>
</tr>
<tr>
<td>Vulnerability</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
<td></td>
</tr>
<tr>
<td>Population</td>
<td>30490</td>
<td>2255</td>
<td>1465</td>
<td>34210</td>
</tr>
<tr>
<td>Specific risk</td>
<td>0.0133</td>
<td>0.0410</td>
<td>0.3133</td>
<td>0.3676</td>
</tr>
</tbody>
</table>

Table 6: Results for the specific risk for population calculated as the product of hazard, vulnerability, and number of persons within a particular hazard class for the 5 different landslide types (Castellanos & van Westen 2008)
From Table 6, it can be concluded that the annual population risk for landslides in Guantánamo province is low (0.8 persons/year). As there are no official records available on landslide casualties it is difficult to validate this at this moment. The method allows also quantifying the risk in monetary values for direct damage to roads, agricultural areas, facilities, and protected areas. The results allow a comparison of annual risks with those from other hazard types, and can form the basis for planning risk reduction measures. In the estimation of the semi-quantitative risk, it is important to keep in mind that there are a number of estimated factors that need to be quantified more in detail in future. These relate specifically to the estimation of temporal probability, and vulnerability. Both require the generation and maintenance of a landslide inventory for the province, which also includes actual damage information. Also a more detailed evaluation of the effect of different landslide magnitudes should be taken into account, as well the use of different return periods for the same landslide type and the inclusion of landslide runout assessment.

**MUNICIPAL SCALE ANALYSIS**

The study area, within the San Antonio del Sur municipality, is located in eastern Cuba (See Figure 4) 60 kilometers from the city of Guantánamo, the capital of the province with the same name. The main access to the area is by the coastal road connecting Guantánamo and the eastern municipalities.

A geomorphological map, including the landslide inventory, at scale 1:50,000 (Figure 11) was prepared from interpretation of two sets of aerial photographs (of 1:25,000 and 1:37,000 scale) and fieldwork. Both photo sets correspond to a national aerial survey carried out at the beginning of the 1970s. The 1:37,000 scale photos (55 in total) cover the whole study area with four flight lines and were taken from 2-Feb-1972 till 19-Mar-1972. The 1:25,000 scale photos (46 in total) cover south-west part in three flight lines and were taken between 5-Dec-1971 and 21-Dec-1971. The photos were interpreted with a TOPCON stereoscope on transparent paper and transferred to digital format by on-screen digitizing over using other image products for double checking (anaglyph, shaded DEM, Landsat TM true color composite and digital topographic map). The photo-interpreted units were checked in the field by three people during a fieldwork campaign which took three weeks. The area was divided into 603 terrain mapping units (TMU). A TMU can be considered a homogeneous mapping unit on the basis of geomorphologic origin, physiography, lithology, morphometry, and soil geography (Meijerink 1988). A single landslide was considered an individual TMU. In certain cases, when the size was large enough, landslide zones such as scarps, bodies and depressions were also considered a separate TMU.

By far the most striking geomorphological feature in the study area is the large oval shaped depression (Puriales de Caujeri valley), which is considered to be a graben with elevation differences up to 500 meters. The valley is limited on the west by a large scarp of the Sierra de Caujeri, with some active retrogressive mass movements. On the southern and northern parts the valley is also surrounding by major fault scarps. The origin of the Puriales de Caujerí depression can be interpreted as a combination of tectonic and mass wasting processes.

The geomorphological mapping provided in-depth knowledge of the causal factors for landslides in the study area and was used to assess landslide susceptibility. Also at this scale, qualitative weighting, one of the heuristic methods, was selected, given the relative small scale, the available data, and the characteristics of the study area. Besides, the TMU mapping
may produce biased results when using a statistical method due to the high spatial correlation between the landslides inventory and some units in the TMU map. The following criteria were used in the susceptibility analysis: Geomorphology, Topography, Geology, Tectonics, and Hydrology. These criteria were further subdivided into nine variables, specific attributes, such as slope, internal relief and slope shape for Topography. The variables are described in Table 7. The variables were standardized and weights were assigned to the corresponding levels of criteria and variables in three different ways: directly by expert opinion, by pairwise comparison matrix and by ranking. The weight values range between 0 and 1 and need to sum up to 1 among the variables within a criterion and among the criteria. For checking the weight assignment, the decision-support system called DEFINITE was used (Janssen & Herwijnen 1994). In the first method, the weights of the criteria and variables were assigned directly based on expert opinion and field experience.

Figure 11: Geomorphological map. San Antonio del Sur, Guantánamo Province, Cuba (Castellanos & van Westen 2007b).
For the pairwise comparison matrix, each variable (or criterion) is compared to all others in pairs in order to evaluate whether they are equally significant, or whether one of them is somewhat more significant / better than the other for the goal concerned. In the ranking method, the criteria and variables are simply ranked according to their importance as landslide controlling factors. The rankings can be considered units on an ordinal scale. Consequently the weights can be found by standardizing the rank order. The three weighting methods gave comparable results, as can be seen from Table 7. For the pairwise comparison matrix method, the inconsistency value was 0.08, demonstrating that the weights are sufficiently reliable. The inconsistency parameter measures randomness of the expert judgments, and ranges from 0 to 1. As a conclusion, the initial weights assigned by expert opinion were taken for the analysis. The final weights of the resulting map ranged from 0.5 to 47.1. This map was classified into 10 divisions interactively, during which the relation with existing landslide areas and geomorphological units was evaluated. Although the map gives a good indication of the qualitative landslide hazard in the study area, too many classes might make it difficult to use by decision makers for development planning. Therefore, the hazard map has ten classes, which are grouped into three simplified categories: high, moderate and low (See Figure 12 and Table 8).

Table 7: Weight for criteria and variables for three methods (Castellanos & van Westen 2007b)

<table>
<thead>
<tr>
<th>Components</th>
<th>Direct Method</th>
<th>Pairwise Matrix</th>
<th>Ranking Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topography</td>
<td>0.3</td>
<td>0.224</td>
<td>0.257</td>
</tr>
<tr>
<td>Slope</td>
<td>0.7</td>
<td>0.7</td>
<td>0.7</td>
</tr>
<tr>
<td>Internal Relief</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Shape</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Geology</td>
<td>0.2</td>
<td>0.131</td>
<td>0.157</td>
</tr>
<tr>
<td>Formation</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Tectonic</td>
<td>0.05</td>
<td>0.040</td>
<td>0.065</td>
</tr>
<tr>
<td>Active faults</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Hydrology</td>
<td>0.05</td>
<td>0.038</td>
<td>0.065</td>
</tr>
<tr>
<td>Springs</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Drainage density</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>Geomorphology</td>
<td>0.4</td>
<td>0.566</td>
<td>0.457</td>
</tr>
<tr>
<td>Subunits</td>
<td>0.4</td>
<td>0.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Landslides zones</td>
<td>0.6</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Total for criteria</td>
<td>1</td>
<td>0.999</td>
<td>1.001</td>
</tr>
</tbody>
</table>
Table 8: Characterization of the 10 landslide hazard classes and 2 flooding hazard classes. (See Figure 12) (Castellanos & van Westen 2007b)

<table>
<thead>
<tr>
<th>Hazard Overall class</th>
<th>Hazard Class</th>
<th>Weight range</th>
<th>Area (ha)</th>
<th>Number of TMU units</th>
<th>Number of landslides</th>
<th>Area of landslides (ha)</th>
<th>Landslide density (%)</th>
<th>General hazard description</th>
</tr>
</thead>
<tbody>
<tr>
<td>LOW</td>
<td>H1</td>
<td>0.50-5.16</td>
<td>8,669</td>
<td>88</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>No landslides expected. Areas can be corridors for mudflows or other intensive mass wasting processes. In some parts small landslides can happen in extreme conditions. The areas are suitable for development projects.</td>
</tr>
<tr>
<td>MODERATE</td>
<td>H2</td>
<td>5.17-9.82</td>
<td>4,176</td>
<td>40</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>MODERATE</td>
<td>H3</td>
<td>9.83–14.48</td>
<td>18,107</td>
<td>125</td>
<td>8</td>
<td>210</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>MODERATE</td>
<td>H4</td>
<td>14.49–19.14</td>
<td>18,361</td>
<td>94</td>
<td>22</td>
<td>338</td>
<td>1.8</td>
<td>Moderate to high possibility of landslides occurrence during intensive or prolonged rainfall. These areas contain most of the existing landslide zones. Most of the landslide materials are unconsolidated and susceptible to being reactivated in smaller proportions. More studies are required for development of the area. Land use changes should be previously studied in relation to landslide hazard problem.</td>
</tr>
<tr>
<td>MODERATE</td>
<td>H5</td>
<td>19.15–23.80</td>
<td>2,013</td>
<td>85</td>
<td>46</td>
<td>901</td>
<td>44.7</td>
<td></td>
</tr>
<tr>
<td>MODERATE</td>
<td>H6</td>
<td>23.81–28.46</td>
<td>1,702</td>
<td>87</td>
<td>79</td>
<td>1,428</td>
<td>83.8</td>
<td></td>
</tr>
<tr>
<td>HIGH</td>
<td>H7</td>
<td>28.47–33.12</td>
<td>969</td>
<td>77</td>
<td>76</td>
<td>963</td>
<td>99.3</td>
<td>High to very high landslide hazard areas. A high possibility of landslide occurrence during rainy conditions. No development is recommended in these areas. Possible relocation of land for agricultural use. Highly recommended re-allocation of existing population in these areas</td>
</tr>
<tr>
<td>HIGH</td>
<td>H8</td>
<td>33.13–37.78</td>
<td>719</td>
<td>59</td>
<td>59</td>
<td>715</td>
<td>99.5</td>
<td></td>
</tr>
<tr>
<td>HIGH</td>
<td>H9</td>
<td>37.79–42.44</td>
<td>291</td>
<td>36</td>
<td>36</td>
<td>289</td>
<td>99.4</td>
<td></td>
</tr>
<tr>
<td>HIGH</td>
<td>H10</td>
<td>42.45–47.10</td>
<td>439</td>
<td>24</td>
<td>24</td>
<td>438</td>
<td>99.8</td>
<td></td>
</tr>
<tr>
<td>FLOODS</td>
<td>F1</td>
<td>N/A</td>
<td>1325</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Flooding areas up to 5 years return period taken from local Civil Defence authority and updated yearly. Appropriate warning system needs to be maintained. The area could be used for seasonal agricultural products. Land-use planning should consider flooding hazard limits to re-allocated existing infrastructure and avoid new developments.</td>
</tr>
<tr>
<td>FLOODS</td>
<td>F2</td>
<td>N/A</td>
<td>503</td>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>Dam break flood limit taken from dam project report. Engineering conditions of the dam should continuously be checked. The area could be used for agricultural products. Land-use planning should consider this hazard limit to re-allocated existing infrastructure and avoid new developments.</td>
</tr>
</tbody>
</table>
Figure 12: Hazard map, San Antonio del Sur, Guantánamo, Cuba. See Table 8 for explanation of the legend (Castellanos & van Westen 2007b)

Too little information was available to carry out a more sophisticated landslide risk assessment. In particular, there was not enough data on the probability of landslides of different magnitudes to make a (semi) quantitative risk assessment. Therefore only a basic qualitative analysis was carried out through the combination of the landslide hazard map with basic elements at risk. During the fieldwork, information was collected on buildings and roads. As the study area is a rural environment, most of the buildings are isolated farmhouses; a number of small schools and medical centres also were identified. Most roads are unpaved
country roads, except for the main road in the south of the study area, which runs along the coast, mostly on the land-side of the coastal hills.

LOCAL SCALE ANALYSIS
This level of analysis was done on two areas at two scales, the escarpment Sierra de Caujerí located in the western border of Caujerí valley and the Jagüeyes landslide in the centre of the escarpment (See Figure 4). It is a tectonic scarp with an average of 500 meter high difference of sub-horizontal limestone and marls. Along the escarpment, about 40 large landslides were mapped, most of them in dormant conditions (See Figure 11). The limestone presents many karstic features and groundwater flows into karstic aquifers. Many springs could be found in the accumulation zone or in the foot of the landslides flowing into the valley drainage system. The landslides are mostly rotational rock slides that end in mud flows at the valley bottom. Different landslide activity distribution can be observed like advancing and retrogressive and landslide activity styles like successive and multiple.

The most catastrophic landslide (Jagüeyes) in the Sierra de Caujerí scarp occurred after three days of heavy rain during the passing of cyclone Flora on October 8, 1963, the most devastating meteorological event known that affected Cuba (See Figure 4). A total of 1,100 mm of rainfall in three days was recorded in the Sierra de Caujerí area. The successive rotational rockslide occurred in two pulses at about 45 minute intervals, which allowed some of the inhabitants to escape, whereas 5-10 others were killed. No technical report was made directly after the event although some data were recorded during the fieldwork when a number of interviews were held with some of the survivors. However, due to the long time since the occurrence of the landslide this information may no longer be reliable. Most of the landslides in the Sierra de Caujerí scarp consist of a large scarp on the upper part, often up to 100 meters high, which has cut almost vertically the limestone layer of the Yateras formation and the underlying Maguey formation. This scarp is actually the back-scarp of multiple landslides, which change from rockslide to debris flows. The area is used mostly for farms, and private and state-owned organizations. There are a few hundreds of farm houses and small primary schools. The road system is poor and unpaved one lane road predominates. Despite the low value of elements at risk, the economical value of the area is important for the province as here is found the main food production for more than 200,000 inhabitants in the province.

A geophysical survey was carried out in order to recognize the underground conditions and detail photo interpretation was made using aerial photos since 1956 until 2000. A DEM was created by digital photogrammetry techniques. A run-out model was applied in Jagüeyes landslide using probabilistic approximations and retrospective risk assessment was evaluated. In this case, element at risk were surveyed as much as possible house by house. With theses results, the risk model was applied to the escarpment for all houses and people found in the area down slope in order to estimate the risk and define risk buffer zones with particular landuse planning considerations as well as risk reduction plan.
Figure 13: Aerial photo interpretation of photos from 1956 (above) and 1972 (below) in which the occurrence of the Jagüeyes landslide can be clearly observed (Castellanos 2008)

Figure 14: Result of the comparison of two DEMs derived from air photos before and after the occurrence of the Jagüeyes landslide (Castellanos 2008)
CONCLUSIONS
There is still no universally applicable methodology for landslide risk assessment, as each country has its own characteristics with respect to organization, available data and environmental situation. For Cuba, we hope that the methodology which is presented here can form the basis for the landslide risk component of the multi-hazard risk assessment method of the Civil Defence. The multi-hazard risk assessment should be carried out in every province by a team previously trained by the national group and working with their own resources. The establishment of a national system for multi-hazard risk assessment for all the municipalities in Cuba also has large implications in terms of the level of standardization required. In this Geographic Information Systems (GIS) and Spatial Data Infrastructure (SDI) are key components. Information technologies in Cuba have been very much marked by the more than 50 years US Embargo, which forced Cuba to find alternative solutions.

The local authorities at different levels are the final users of the risk assessment. They are responsible for managing the risk supported by the Civil Defence specialists, other organizations and the local communities. As ordered in 2005 by the vice-president of the National Council of Defence, the Ministry of Science, Technology and Environment (CITMA) is responsible for making the risk assessment in the country and the Agency of Environment (AMA) was enrolled in this task to create a multi-disciplinary group on risk assessment. This group started to make risk assessments for flooding, strong wind and sea surges in the municipalities of Havana City and they created a methodology to generalize these studies in the rest of the country (AMA 2007). It is not defined how other disasters will be introduced in this local multi-hazard risk assessment. CITMA specialists and other local researchers in the provinces and municipalities, after trained, are conducting risk assessments in their territories. As they know the area better, they are responsible for data collection as well. Data forms have been created in order to standardize this process. It is expected in the coming years to complete an appropriate set up for data collection including the identification of users incorporating the local centres for risk management.

One of the key components in the data collection for landslides will be historical data. Historical landslide information is not available, except for a few isolated landslides. A national landslide inventory system is now under development. An important component of this system will be the involvement of local staff of the Civil Defense at the 169 municipal centres. A simple landslide reporting form has been designed, and workshops will be conducted to train the staff and make them aware of the procedure. Once the local officers report a landslide, a landslide expert from the central office will visit the site and complete the questionnaire in more detail.

The aim is also to link the landslide risk assessment with the early warning system, which is well developed for hurricanes, and related flooding. Here also rainfall estimates from satellite imagery can be used, such as from Tropical Rainfall Measuring Mission (TRMM) and Multi-satellite Precipitation Analysis (TMPA), which is used to issue landslide warnings based on a threshold value derived from earlier published intensity-duration-frequency relationships for different countries (Hong et al. 2007). Hong & Adler (2007) propose an early warning system for global landslide warnings, based on the TRMM rainfall estimations, combined with the near-real time ground shaking prediction system for earthquakes (Wald et al. 2003) and with generalized landslide susceptibility information, including altitude information from SRTM, and landcover information, derived from MODIS.
REFERENCES
Agencia de Medio Ambiente (AMA) (2007). Lineamientos metodológicos para la realización de estudios de peligro, vulnerabilidad y riesgo de desastres de inundaciones por penetraciones del mar, inundación por intensas lluvias y afectaciones por fuertes vientos., Agencia de Medio Ambiente (AMA), Ministerio de Ciencia, Tecnología y Medio Ambiente (CTIMA), La Habana.


ACKNOWLEDGEMENTS

This study was supported by the national landslide risk assessment programme of the Cuban Civil Defence Organisation. This study was also carried out as a component of the SLARIM research project, within the United Nations University - ITC School for Disaster Geo-Information Management. We would like to express to the National Civil Defence our gratitude for their contribution.
REPORT ON INNOVATIVE AND DIGITAL TECHNOLOGY

Suzanne Lacasse  
International Centre for Geohazards  
Norwegian Geotechnical Institute, Oslo, Norway

Abstract: Session 3 included eight technical presentations on innovative technology and a discussion on landslide prediction, remote sensing, digital mapping and early warning system development. An ensuing plenum discussion controlled the conclusions reached, verified consensus and discussed future strategic directions.

INTRODUCTION
Session 3 provided eight technical presentations on innovative technology, covering hazard management, GIS and monitoring. The papers in the proceedings gave examples of:

▪ Use of slope sensors to assess rock stability and as an aid for hazard management  
▪ Real-time monitoring to predict slope stability (2 papers)  
▪ Recent development and application of geomatics for risk management in Hong Kong  
▪ Time-domain reflectometry for control of soil nailing work  
▪ Scatters and web GIS for monitoring of landslides  
▪ Early warning system for rainfall-induced landslides  
▪ Multi-scale landslide risk assessment in Cuba

The discussion session was done in four break-out sessions, each led by two discussion leaders. Each group was given one topic to discuss, and was asked to reach consensus and prepare a set of recommendations. After reassembly in plenum, the consensus results from the four groups were presented and the conclusions were verified by all participants in the session. Future directions within innovative technologies in landslide hazard assessment and risk management were discussed.

The four topics debated were:

▪ Landslide Prediction (Discussion Leaders: Giovanni Crosta and Ken Ho)

Landslide evaluation and prediction has several stages, including: (i) detection of movement through monitoring, (ii) temporal evaluation through analysis of data and numerical simulation, and (iii) definition of thresholds identifying critical instability. How should these stages affect landslide prediction and risk assessment approaches?

▪ Remote Sensing (Discussion Leaders: Michel Jaboyedoff and D. Jean Hutchinson)

Are remote sensing tools, such as LiDAR, InSAR or other imagery, practical, affordable tools in landslide hazard assessment programs? What are the strengths and limitations of those techniques?
Digital Mapping (Discussion Leaders: Jordi Corominas and Cees van Westen)

What is the role of landslide susceptibility and risk mapping in controlling landslide hazards in different countries or regions, considering issues of data scale and availability, the regulatory framework, and the frequency and impact of previous landslides?

Early Warning System Development (Discussion Leaders: Lars Blikra and Mike Winter)

What is required to create a true ‘early warning system’ (EWS), as opposed to a monitoring program? What are the innovations as well as the challenges with EWS, and what is the future direction expected to be?

Ten persons chose to work on Topic 1 (landslide prediction), five selected Topic 2 (remote sensing), five selected Topic 3 (digital mapping) and nine chose Topic 4 (early warning system development). The following summarises the conclusions reached in each discussion group, and the main discussion points.

**LANDSLIDE PREDICTION**

*Landslide evaluation and prediction has several stages, including: (i) detection of movement through monitoring, (ii) temporal evaluation through analysis of data and numerical simulation, and (iii) definition of thresholds identifying critical instability. How should these stages affect landslide prediction and risk assessment approaches?*

For site-specific slopes with monitoring by instruments, the slope displacement represents one of the key indicators of actual slope performance. The monitoring may include: in-situ, remote, and surface/subsurface methods. It is important to consider the issues of ‘what to measure’ and ‘where to measure’, system uncertainty and reliability etc, in order to avoid misleading results.

**Monitoring**

The objectives of the monitoring of the movement include (1) monitoring for public safety and risk management, (2) health or performance monitoring, (3) regional warning (e.g. landslide), (4) construction quality control and (5) the understanding of the behaviour observed (technical development). Different purposes will have different monitoring and follow-up requirements. One needs to consider the likely modes of failure in data interpretation and the setting of threshold values (e.g. brittle versus ductile failure controlled, among others, by material and mass properties).

The detection of movement is a more direct measure of the potential instability than the other measurements. If only pore pressures are monitored, it may be difficult to foretell how imminently the instability may occur. It is however best to relate the movement monitored to other monitoring data (e.g. pore pressure, rainfall, etc.) to have a more complete appreciation of the slope behaviour.

**Interpretation of Movement Data**

The interpretation of movement data needs a suitable model for projection of behaviour, e.g. the observational approach, which is typically done empirically or based on common sense instead of theoretical/numerical analyses.
Numerical simulation can be used to set-up a framework to interpret ground movement; with the observed data used to calibrate the model predicting the slope behaviour up to failure. It is best to do the calibration against a slope that has gone to failure. The code will differ from conventional limit equilibrium stability calculations, but will be more complex, and require more input parameters and hence have more uncertainty.

Thresholds need to relate to anticipated mode of failure and time for response. In the case of a brittle failure with little warning signs before collapse with fast-moving debris, the movement monitoring could give result to a false sense of security! One also needs to consider whether there are potential mechanisms under which ductile failures could become brittle failures. Where a slope failure is brittle, an observation of no movement could give a false sense of security.

**Alert and Action Levels**
There is the need for more than one alert or action level, related to the assessed/judged probability of failure and the time to develop failure. It is difficult to come up with absolute limits on tolerable slope movement because of the lack of experience. The rate of change of movement is usually of the essential and central focus of the measurements and warning. One must allow sufficient time for response. Risk communication is also essential if monitoring is to be used for risk management decisions.

**Risk Communication**
The issues of false alarms and loss of credibility remain an issue. Based on past experience in Hong Kong, the use of movement monitoring results may be more effective than measurements of groundwater pressure. There are more frequent false alarms when threshold groundwater pressure values are reached because of conservative pore pressure assumptions made in the slope stability analyses.

**REMOTE SENSING**
Are remote sensing tools, such as LiDAR, InSAR or other imagery, practical, affordable tools in landslide hazard assessment programs? What are the strengths and limitations of those techniques?

The group prepared a summary table with the strengths, limitations, resolution of existing remote sensing technologies today (Table 1). Future directions for each remote sensing system are also included. Issues of “where”, “when”, “why” and “costs” influence the selection of technique(s) in practice. In plenum, the session participants agreed with the table presented and the evaluations made.

**DIGITAL MAPPING**
What is the role of landslide susceptibility and risk mapping in controlling landslide hazards in different countries or regions, considering issues of data scale and availability, the regulatory framework, and the frequency and impact of previous landslides?

**Regulatory Framework**
The group chose to rephrase the question to: How can susceptibility, hazard and risk maps help in risk management? The GIS approach can obviously help. The mapping will help even if there is no regulatory framework, and even if there is no formal planning, with enhanced
awareness and community based measures. GEO in Hong Kong is a good example. Where GEO is an authority and can impose decisions.

Table 1: Remote sensing technologies, strength, limitations and resolution

<table>
<thead>
<tr>
<th>System</th>
<th>Applications</th>
<th>Resolution</th>
<th>Limitations</th>
<th>Strengths - DEM from all Future</th>
</tr>
</thead>
<tbody>
<tr>
<td>Photo-grammetry</td>
<td>Terrestrial joint survey, scarp and change detection</td>
<td>cm to m, depending on scale</td>
<td>field of view, image resolution, requirement for surveyed positions, vegetation obscurance</td>
<td>rapid, cheap, long term record, stereoscopic - build DEMs and see terrain conditions</td>
</tr>
<tr>
<td></td>
<td>Airborne landslide inventory and time series analysis</td>
<td></td>
<td></td>
<td>digital imagery, use of high resolution scanners and video</td>
</tr>
<tr>
<td>LIDAR</td>
<td>Ground based - static landslide monitoring, landslide mapping, topography extraction HR joint surveys, moisture detection</td>
<td>mm to cm</td>
<td>humidity, distance limitations, angular limitations, reflectivity, vegetation obscuration, too much data?, field of view</td>
<td>multi-return LIDAR allows bare earth model, very high accuracy, high rate of acquisition, perspective views possible</td>
</tr>
<tr>
<td></td>
<td>Ground based - mobile HR, joint surveys, moisture detection</td>
<td></td>
<td></td>
<td>costs will decrease, signal analysis will improve, automated feature extraction?</td>
</tr>
<tr>
<td></td>
<td>Airborne - fixed wing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Airborne - helicopter</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>InSAR</td>
<td>Standard movement detection and monitoring, time series displacement, HR movement detection (ground based)</td>
<td>mm</td>
<td>visibility and shadowing, need reflectance, doesn’t penetrate vegetation, limited to slow movements</td>
<td>large area survey, long term monitoring, return frequency affects use, comparison or combination of ascending and descending paths, movement measurement, rapid mapping of targeted areas</td>
</tr>
<tr>
<td></td>
<td>PS</td>
<td>as above, plus only works with stable reflectors, very expensive</td>
<td>as above, plus mm accuracy movement detection</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ground based</td>
<td>as above, but can monitor faster movements, but semi-permanent installation and view point is required</td>
<td>as above, plus can pick up larger movements</td>
<td></td>
</tr>
<tr>
<td>Optical satellite images</td>
<td>landslide inventory and time series analysis 10's cm to D11 10's m</td>
<td>low resolution</td>
<td>vegetation classification, water content, change detection</td>
<td>higher resolution satellites to be deployed, more satellites will increase coverage and frequency</td>
</tr>
<tr>
<td>Weather Radar</td>
<td>precipitation intensity, early warning from rainfall intensity and accumulation</td>
<td>Depending on calibration, km</td>
<td>Calibration is required</td>
<td>helps map spatial distribution of weather</td>
</tr>
<tr>
<td>Others</td>
<td>Thermal, IR</td>
<td></td>
<td></td>
<td>enhanced mapping of weather systems, cheaper systems</td>
</tr>
</tbody>
</table>

Objectives of Mapping
Mapping should be available to enable landslide risk management to be effective. The mapping can consist of (1) susceptibility maps for general land-use planning; (2) hazard maps, which give more information and allow for finer tuning of risk management; (3) risk maps, forming the basis for disaster preparedness and early warning; and (4) risk maps for risk-based design of remedial measures.

Technical Aspects of Mapping
Susceptibility mapping is possible and useful, both in terms of source area and run-out area. Establishing hazard maps is difficult, establishing risk is even more difficult. Landslide inventory is essential, and vulnerability information is usually where there is most information missing.

There are interdisciplinary issues: Too much is today looked at by earth scientists and geotechnical engineers, whereas vulnerability is much more than just the physical vulnerability.

The mapping scale depends on the objectives of the mapping: scale of study, available data, techniques and models used and whether the mapping is static or dynamic (dynamic meaning
that the data change with time). At the site investigation level, the mapping seems less useful and can be replaced by engineering design.

The scale of a susceptibility map is dictated by the constraints of the terrain and quality of data. For instance, if the quality of data is not too reliable, only small-scale maps can be prepared. Depending on the scale wanted, one has to choose the adequate data needed.

**Uncertainty**
There are many uncertainties in susceptibility, hazard and risk mapping that need to be dealt with, including landslide inventory, input data (geology, geomorphology, terrain, slope inclination, soil layer thickness, properties of soils), models and methods used and temporal changes for the elements at risk. In fact, vulnerability and hazard change continuously.

The sound and correct sequence in the mapping process is:

Susceptibility → Hazard → Vulnerability and Risk.

An example of the data and aspects to be considered is given in Table 2.

In considering the vulnerability of a facility, other types of hazard should also be considered in addition to the landslide hazard. Multi-hazard risk is important. Furthermore, other aspects of vulnerability (not only physical, but also social, economical and environmental should be addressed.

**EARLY WARNING SYSTEM DEVELOPMENT**

*What is required to create a true ‘early warning system’, as opposed to a monitoring program? What are the innovations as well as the challenges with EWS, and what is the future direction expected to be?*

The group concentrated its discussions on the what is required to create a true ‘early warning system’, including how long a warning is needed, what needs to be achieved in the warning period, what are the innovations as well as the challenges with EWS, and what are the future directions expected to be for EWS.

**Requirements for a “True” Early Warning System**

As opposed to a monitoring programme, a “true”, reliable early warning system needs:

- Understanding of the phenomena – e.g. landslide
- Historical knowledge, e.g. rainfall triggers
- Effective and appropriate monitoring programme
- Interpretation of data
- Decision-making
- Ability to use a ‘human’ that can override
- Public tolerance of false alarms
- System for informing the authorities
- Pre-planned action plans for effective and timely implementation
- Feedback loop
- Evolution of knowledge (driving the system) and thus the system itself, in other words “adaptability”
The length of time needed for a landslide warning depends on the landslide behaviour. What needs to be achieved in a warning includes, road for closure, e.g. for cars moving at 100km/h over at length of perhaps 5km. For larger populations, much more time is needed.

If a landslide occurs or is on the verge of occurring, time is needed for detection through the EWS, notification (authorities - police, city council- required action (closure of roads, evacuation, ...). Societal needs and controls are also a factor. But utmost, communication is the most critical need.

Sharing information on the monitoring system is an absolute requirement for an effective early warning system. The expectation from the general public and the regulator is different. One difficulty with the monitoring is the efficient sharing of information. There is generally very little time for the regulator to disseminate monitoring information. Communication also depends on public tolerance, method used to share the information and the measurements themselves and the perception of their reliability.

**Innovations and Challenges with EWS**
The technology exists today, both the instruments, systems and models. The profession needs to make all this knowledge work together. It is not the State-of-the-Art that is the problem, it is the application of the State-of-the-Art that needs to be improved.
Future directions are expected to (1) make nature simpler, i.e. simple visual observation in the field, improved accessibility and increasingly rapid communication; (2) education and awareness; (3) training of selected public, including having then input information (but this needs to be treated carefully); and (4) one cannot put sensors everywhere, so one needs to find interpolating solutions.

ACKNOWLEDGEMENT
The author wishes to thank Messrs W.K. Pun, of GEO and José Cepeda, of ICG NGI, for their careful note-taking during the session and the Organizing committee for the recording of the different contributions to the session. Professor Jean Hutchinson’s assistance with the organization of the session is also gratefully recognized. The contributions of the eight discussion leaders, Professor Giovanni Crosta, Mr. Ken Ho, Professor Jean Hutchinson, Professor Jordi Corominas, Professor Michel Jaboyedoff, Professor Cees van Westen, and Dr. Lars Harald Blikra, in guiding and reaching consensus in the break-out session were invaluable.
The 2007 International Forum on Landslide Disaster Management

Edited by:

Ken Ho
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Victor Li
Victor Li & Associates Ltd., Hong Kong

Volume II
Cover photographs:
The 1990 Tsing Shan channelised debris flow in Hong Kong (left) and three-dimensional landslide mobility modelling (right).

The copyright is owned by the authors. No part of this publication or the information contained herein may be reproduced, stored in a retrieval system, or transmitted in any form or by any means, electronic, mechanical, by photocopying, recording or otherwise, without prior written permission from the authors.

Although all care is taken to ensure the integrity and quality of this publication and the information herein, no responsibility is assumed by the publishers nor the authors for any damage to property or persons as a result of use of this publication and/or the information contained herein.

Published by:
Geotechnical Division, The Hong Kong Institution of Engineers


Printed in Hong Kong
TABLE OF CONTENTS

Foreword VII
Organisation IX
Sponsors XI
Technical Programme XIII

VOLUME I

Opening Address 1
R.K.S. Chan

Session 1  Country / Regional Reports on Landslide Risk Management Practice

Development of slope management in Malaysia 3
C.H. Abdullah, A. Mohamad, M.A.M. Yusof, S.S. Gue & M. Mahmud

Landslide risk management in Hong Kong 17
R.K.S. Chan & S.H. Mak

Experience on landslide risk management in the Eastern Pyrenees (Spain and Andorra): Achievements and challenges 49
J. Corominas

The Alerta Rio System 71
R.N. d’Orsi

Report on landslide impacts and practices in Switzerland: Need for new risk assessment strategies 79
M. Jaboyedoff & Ch. Bonnard

Landslide risk management: Country Report for Norway 99
S. Lacasse & F. Nadim

Slope safety and landslide risk management in Korea 125
S.G. Lee & S.R. Hencher

Landslide risk management in France: Principles, organisation and challenges 169
E. Leroi

Landslide risk management: Country Report - Australia, 2007 205
A. Leventhal

Evolution of slope-land hazard mitigation strategies and measures in Taiwan 235
M.L. Lin & H.L. Wu

Landslide hazard activities in the United States 251
P.T. Lyttle
Challenges in landslide risk management in a European perspective
*D. Patel & O. Kjekstad*

Landslide disaster management in Italy
*L. Picarelli, P. Versace, R. de Riso & M. Palmieri*

Country Report from Japan: Progress of landslide dynamics and the International Programme on Landslides
*K. Sassa*

Landslide risk management in the United Kingdom
*M.G. Winter, R.G. McInnes & E.N. Bromhead*

Landslide mitigation strategy and implementation in China
*Y.P. Yin, S.J. Wang & Z.Y. Chen*

Session Chair’s report on landslide risk management practice
*A.W. Malone*

Record of discussion
*J.C.Y. Cheuk, H.W. Sun & J.W.C. Lau*

**Session 2  Forensic Landslide Investigations**

The investigation of the Aznalcóllar dam slide failure
*A. Gens & E. Alonso*

Reflections on forensic investigations to determine the nature, risk, and causes of the 1979 Abbotsford landslide, Dunedin, New Zealand
*G.T. Hancox*

Repeated collapse of cut slopes despite remedial works
*S.G. Lee & S.R. Hencher*

The Storegga slide - case study of an offshore megaslide in the Norwegian Sea
*F. Nadim & T.J. Kvalstad*

Landmark landslides in Malaysia
*M.A. Othman, M. Mahmud, A. Mohamad & M.J. Sulaiman*

Landslide investigations and risk mitigation: The Sarno case
*P. Versace, G. Capparelli & L. Picarelli*

Insights from some milestone landslides in Hong Kong
*H.N. Wong & K.K.S. Ho*

Landslide hazard monitoring and warning system for Li-shan area
*H.L. Wu & M.B. Su*

Session Chair’s report on forensic landslide investigations
*L. Picarelli*

Record of discussion
*J.S.H. Kwan & H.W. Sun*
Session 3  Innovative and Digital Technology

Predicting slope failure using real-time monitoring technology and the TRS sensor
K.T. Chang, H.S. Han, J.F. Wang, A.N.L. Ho & D. Mothersille  
579

From permanent scatterers to WebGIS: Innovative techniques for landslide monitoring and early warning
G. Falorni, P. Canuti, P. Farina, L. Leoni & N. Casagli  
587

Assessment of slope sensor data to support rock slope stability analysis and infrastructure hazard management
D.J. Hutchinson, M. Diederichs, K. Kalenchuk & M. Lato  
599

Development and application of geoinformatics for landslide risk management in Hong Kong
K.C. Ng & H.N. Wong  
619

Prediction of rainfall-induced landslides in unsaturated granular soils for setting up of early warning systems
L. Picarelli, P. Versace, L. Olivares & E. Damiano  
643

Application of time domain reflectometry for quality control of soil nailing works
667

Advances in real-time monitoring of slope stability
J.M. Strout, E. DiBiagio & R.G. Omli  
687

Development of a method for multi-scale landslide risk assessment in Cuba
C.J. van Westen & E.A. Castellanos Abella  
717

Session Chair’s report on innovative and digital technology
S. Lacasse  
747

VOLUME II

Session 4  Benchmarking Exercise on Landslide Debris Runout and Mobility Modelling

Review of benchmarking exercise on landslide debris runout and mobility modelling
O. Hungr, N.R. Morgenstern & H.N. Wong  
755

The 2005 Tate’s Cairn debris flow: Back-analysis, forward predictions and a sensitivity analysis
J. Cepeda  
813

Analysis of Hong Kong debris flow with an energy based model
D. Chan, N.R. Morgenstern, D. Tran & X.B. Wang  
835

Landslide mobility analysis using MADflow
J.H. Chen & C.F. Lee  
857

Approach to numerical modelling of long runout landslides
G.B. Crosta, S. Imposimato & D. Roddeman  
875
Benchmarking TITAN2D mass flow model against a sand flow experiment and the 1903 Frank slide
S. Galas, K. Dalbey, D. Kumar, A. Patra & M. Sheridan

Two models for analysis of landslide motion: Application to the 2007 Hong Kong benchmarking exercises
O. Hungr, M. McKinnon & S. McDougall

Application of 2D-finite volume code FLATModel to landslide runout benchmarking exercises
M. Hürlimann, V. Medina & A. Bateman

Benchmarking exercise on landslide mobility modelling - runout analyses using 3DMM
J.S.H. Kwan & H.W. Sun

Benchmark exercises for granular flows
A. Lucas, A. Mangeney, F. Bouchut, M.-O. Bristeau & D. Mège

A SPH depth integrated model with pore pressure coupling for fast landslides and related phenomena

A set of benchmark tests to assess the performance of a continuum mechanics depth-integrated model
M. Pirulli & C. Scavia

Landslide detachment mechanisms: An overview of their mechanical models
R. Poisel & A. Preh

Punta Thurwieser rock avalanche and Frank slide: A comparison based on PFC3D runout models
R. Poisel, A. Preh & O. Koc

Landslide simulation by geotechnical model adopting a model for variable apparent friction coefficient
F.W. Wang & K. Sassa

Report on benchmarking exercise of landslide debris runout and mobility modelling
J.S.H. Kwan & H.W. Sun

Closing Address
L. Picarelli

Photographs

Register of Participants

Author Index
FOREWORD

The International Forum on Landslide Disaster Management was held in Hong Kong in December 2007 to commemorate the 30th anniversary of the implementation of landslide risk management by the Geotechnical Engineering Office, Government of the Hong Kong Special Administrative Region (SAR). The event was convened under the auspices of the Joint Technical Committee on Landslides and Engineered Slopes (JTC-1) of the ISSMGE, ISRM and IAEG.

The Forum has gathered together some 60 participants, who have all been invited by virtue of their expertise in slope engineering, to share their experience and insights. The papers contained in this set of Proceedings have been contributed by eminent international landslide experts. Also included in the proceedings is a record of the discussions and speeches made. The accompanying CD contains the papers as well as the presentation materials.

The papers are arranged under 4 sessions. The first session comprises country or regional reports on landslide risk management practice. The second session covers forensic landslide investigations whilst the third session is on the use of innovative and digital technology in slope engineering. The last session consists of a benchmarking exercise on landslide mobility modelling, which was steered by a Review Committee comprising Norbert Morgenstern, Oldrich Hungr and HN Wong.

The Forum was jointly organised by the Geotechnical Division of the Hong Kong Institution of Engineers, the Geotechnical Engineering Office of the Government of the Hong Kong SAR and the Hong Kong Geotechnical Society. The Organising Committee wishes to express its gratitude to these organisations and to the sponsors named in the volume.

Ken Ho
Chairman, Organising Committee
Hong Kong

October 2008
# ORGANISATION

**Organising Committee**

- **Chairman**
  - Ken Ho

- **Members**
  - Johnny Cheuk
  - Albert Ho
  - May Ho
  - Y C Koo
  - David Kwok
  - Rachel Law
  - Alex Li
  - Eric Li
  - Victor Li
  - L M Mak
  - Charles Ng
  - Loretta Pau
  - W K Pun
  - H W Sun
  - George Tham
  - Jenny Yeung
  - J H Yin
  - Ringo Yu

**International Advisory Committee**

- Robin Fell
- Suzanne Lacasse
- Willy Lacerda
- C F Lee
- Norbert Morgenstern
- Luciano Picarelli

**Review Committee for Benchmarking Exercise on Landslide Runout Analysis**

- Oldrich Hungr
- Norbert Morgenstern
- H N Wong
SPONSORS

China Geo-Engineering Corporation
C M Wong & Associates Ltd.
Fraser Construction Company Ltd.
Fugro (Hong Kong) Ltd.
Fuk Shing Engineering Company Ltd.
Halcrow China Ltd.
LMM Consulting Engineers Ltd.
Maunsell Geotechnical Services Ltd.
New Concepts Engineering Development Ltd.
Ove Arup & Partners Hong Kong Ltd.
Tysan Foundation Ltd.
Victor Li & Associates Ltd.
TECHNICAL PROGRAMME

Monday, 10 December 2007

08:30   Opening Address  
        R.K.S. Chan

Session 1  Country / Regional Reports on Landslide Risk Management Practice  
            (Chair: A.W. Malone)

        A. Leventhal
        Landslide Mitigation Strategy and Implementation in China  
        Y.P. Yin
        Landslide Risk Management in Hong Kong  
        R.K.S. Chan
        Evolution of Slope-land Hazard Mitigation Strategies and Measures in Taiwan  
        M.L. Lin
        Country Report from Japan: Progress of Landslide Dynamics & the International  
        Programme on Landslides  
        K. Sassa

10:30   Coffee Break

10:50   Slope Safety and Landslide Risk Management in Korea  
        S.G. Lee
        Development of Slope Management in Malaysia  
        A. Mohamad
        Landslide Risk Management: Country Report – Canada  
        J. Hutchinson

11:50   Panel Discussion

12:30   Lunch

14:00   Landslide Hazard Activities in the United States  
        P.T. Lyttle
        Challenges in Landslide Risk Management in a European Perspective  
        F. Nadim
        Landslide Risk Management in France: Principles, Organisation and Challenges  
        E. Leroi
        Landslide Disaster Management in Italy  
        L. Picarelli
        Landslide Risk Management: Country Report for Norway  
        S. Lacasse
15:40 Coffee Break
16:10 Experience on Landslide Risk Management in the Eastern Pyrenees (Spain and Andorra): Achievements and Challenges
  J. Corominas
  M. Jaboyedoff
Landslide Risk Management in the United Kingdom
  M.G. Winter & R.G. McInnes
17:10 Panel Discussion
18:30 End of Session

Tuesday, 11 December 2007

Session 2 Forensic Landslide Investigations
  (Chair: L. Picarelli)
08:30 The Investigation of the Aznalcóllar Dam Slide Failure
  A. Gens
The Storegga Slide – Case Study of an Offshore Megaslide in the Norwegian Sea
  F. Nadim
Landslide Hazard Monitoring and Warning System for Li-shan Area
  M.B. Su
Repeated Collapse of Cut Slopes Despite Remedial Works
  S.G. Lee
09:55 Panel Discussion
10:30 Coffee Break
10:50 Landmark Landslides in Malaysia
  A. Othman
Landslide Investigations and Risk Mitigation: The Sarno Case
  P. Versace
Insights from Some Milestone Landslides in Hong Kong
  H.N. Wong
11:55 Panel Discussion
12:30 Lunch
Session 3  Innovative and Digital Technology  
(Chair: S. Lacasse)

14:00  Assessment of Slope Sensor Data to Support Rock Slope Stability Analysis and Infrastructure Hazard Management  
J. Hutchinson

Advances in Real-time Monitoring of Slope Stability  
R.G. Omli

Development and Application of Geoinformatics for Landslide Risk Management in Hong Kong  
K.C. Ng

Predicting Slope Failure Using Real-time Monitoring Technology and the TRS Sensor  
K.T. Chang

Application of Time Domain Reflectometry for Quality Control of Soil Nailing Works  
W.K. Pun

From Permanent Scatterers to WebGIS: Innovative Techniques for Landslide Monitoring and Early Warning  
G. Falorni

15:50  Coffee Break

16:10  Prediction of Rainfall-induced Landslides in Unsaturated Granular Soils for Setting Up of Early Warning Systems  
L. Picarelli

Development of a Method for Multi-scale Landslide Risk Assessment in Cuba  
C.J. van Westen

16:40  Group Discussion

17:30  Plenum Discussion

18:30  End of Session

Wednesday, 12 December 2007

Session 4  Benchmarking Exercise on Landslide Debris Runout and Mobility Modelling  
(Chairs: O. Hungr, H.N. Wong & H.W. Sun)

08:30  Overview on Benchmarking Exercise  
H.N. Wong, H.W. Sun & O. Hungr

09:00  Benchmarking Exercise – Presentations  
J.H. Chen  
O. Hungr  
M. Pastor  
J.S.H. Kwan
M. Pirulli
M. Sheridan

10:30 Discussion
10:50 Coffee Break
11:10 Benchmarking Exercise – Presentations
   K. Sassa & F.W. Wang
   A. Lucas
   J. Cepeda
   D. Chan
   R. Poisel
   G. Crosta
   H.W. Sun (on behalf of M. Hürlimann)
12:40 Discussion
13:00 Lunch cum Exhibition on Digital Technology
15:00 Summary of Benchmarking Exercise Results
   O. Hungr & H.N. Wong
15:30 Discussion
16:10 Closing Address
   L. Picarelli
16:30 Exhibition on Digital Technology
17:30 End of Programme
Session 4  Benchmarking Exercise on Landslide Debris Runout and Mobility Modelling
REVIEW OF BENCHMARKING EXERCISE ON LANDSLIDE DEBRIS RUNOUT AND MOBILITY MODELLING

Oldrich Hungr  
Department of Earth and Ocean Sciences  
University of British Columbia, Vancouver, Canada

Norbert Morgenstern  
Department of Civil and Environmental Engineering  
The University of Alberta, Canada

H. N. Wong  
Geotechnical Engineering Office, Civil Engineering and Development Department  
Government of the Hong Kong Special Administrative Region

OVERVIEW

Introduction
Day 3 of the International Forum on Landslide Disaster Management was devoted to presentation and discussion of the Benchmarking Exercise on Landslide Runout Analysis (referred to the Benchmarking Exercise hereafter). The exercise is aimed at assessing whether the emerging field of landslide debris mobility is on its way towards establishing some degree of commonality among different methods used by various parties, taking stock of the progress made and issues to be further addressed, and facilitating interaction among researchers and practitioners.

In this Benchmarking Exercise, participants completed numerical modelling of selected benchmark cases and provided their reports before the Forum. During the Forum, the participants presented key summaries of their findings and took part in discussions on various aspects of dynamic modelling of landslide mobility. The Terms of Reference of the Benchmarking Exercise is included in Appendix A. Professor N R Morgenstern of the University of Alberta, Professor O Hungr of the University of British Columbia and Mr H N Wong of the Geotechnical Engineering Office were on the Review Panel for the Benchmarking Exercise. The Review Panel was supported by a Hong Kong support group, which assisted in extracting, analysing and summarising the benchmarking results. The support team comprised Ms F W Y Ko, Dr J S H Kwan, Dr H W Sun and Mr K C Wong of the Geotechnical Engineering Office.

Twenty-one research groups working on the subject were invited in March 2007 to participate in the Benchmarking Exercise to assemble dynamic numerical models for a total of twelve cases. The cases are placed in three groups, as follows:

- Group A - Verification test cases
  - (1) Dam-break scenario
  - (2) Laboratory test of dry sand flow prepared by the Swiss Federal Institute of Technology, Lausanne
(3) USGS flume test

- Group B - Debris avalanche/debris flow cases
  - (4) Shum Wan Landslide, Hong Kong
  - (5) Fei Tsui Road Landslide, Hong Kong
  - (6) Sham Tseng San Tsuen Debris Flow, Hong Kong
  - (7) 1990 Tsing Shan Debris Flow, Hong Kong

- Group C - Rock avalanche and debris flood cases and a prediction case
  - (8) Frank Slide, Canada
  - (9) Thurwieser Rock Avalanche, Italy
  - (10) 2000 Tsing Shan Debris Flow, Hong Kong
  - (11) Tate’s Cairn Landslide, Hong Kong
  - (12) Lo Wai Debris Flood, Hong Kong

Packages of input materials were distributed to the interested parties in April 2007. By September 2007, a total of 13 teams (from Austria, Canada, France, Netherlands, Hong Kong, Italy, Japan, Norway, Spain and USA) have submitted their modelling results using their respective numerical methods and rheological models. The participating groups were asked to back-analyse the cases using their models, so as to yield the best simulation of observed behaviour. A forward prediction of one potential landslide (Tate’s Cairn, Hong Kong) was also included.

This report summarises the submissions made by the various teams on the cases that they have attempted and the methods of analysis, including the rheological models and numerical methods that have been adopted, reviews of the modelling results to diagnose notable common points and discrepancies among them. The report then examines the key issues in relation to application of mobility modelling to landslide risk management and identifies potential areas that warrant further research and development.

**Summary of Submissions**

**Benchmarking Results and Models Adopted**

Thirteen groups have submitted their modelling results using their respective numerical methods (Table 1). The cases that have been attempted by the respective groups are listed in Table 2.

The submissions have been summarised in terms of the following key areas of modelling approaches, as detailed in Tables 3(a) to (g):

(a) basic solution approach;
(b) solution dimensions;
(c) solution reference frame;
(d) basal rheology;
(e) internal stress and energy dissipation (other than basal) assumptions;
(f) entrainment of material along flow path; and
(g) variation of basal strength along flow path.
Table 1: List of participants and numerical models used

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Denoted as</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Wang</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>MADFLOW</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>TOCHNOG</td>
</tr>
<tr>
<td>Norwegian Geotechnical Institute (NGI)</td>
<td>RAMMS</td>
<td>RAMMS</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>DAN3D(NGI)</td>
</tr>
<tr>
<td></td>
<td>FLO-2D</td>
<td>FLO-2D(NGI)</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>FLATMODEL</td>
</tr>
<tr>
<td>Geotechnical Engineering Office (GEO), Hong Kong</td>
<td>3dDMM</td>
<td>3dDMM</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>SHALTOP-2D</td>
</tr>
<tr>
<td></td>
<td>RASH3D</td>
<td>RASH3D(Paris)</td>
</tr>
<tr>
<td>University of British Columbia (UBC), Vancouver</td>
<td>DAN3D</td>
<td>DAN3D</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>DAN3D</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>Pastor</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>PFC</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>Sassa-Wang</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>RASH3D</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>TITAN2D</td>
</tr>
</tbody>
</table>
Table 2: Cases included in the submissions by participants

<table>
<thead>
<tr>
<th>Team</th>
<th>Model Denoted As</th>
<th>Group A</th>
<th>Group B</th>
<th>Group C</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️</td>
</tr>
<tr>
<td>University of Milan</td>
<td>TOCHNOG</td>
<td>✔️</td>
<td></td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td></td>
<td>✔️</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td>✔️</td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
<td>✔️</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️</td>
</tr>
<tr>
<td>Université Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>RASH3D(Paris)</td>
<td>✔️</td>
<td></td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td></td>
<td>DAN</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️ ✔️ ✔️</td>
<td>✔️</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>✔️</td>
<td></td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️ ✔️ ✔️</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>✔️</td>
<td>✔️</td>
<td>✔️</td>
</tr>
</tbody>
</table>
Table 3(a): Summary of method of analysis - basic solution approach

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Continuum</th>
<th>Discrete Particulate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Differential(1)</td>
<td>Integrated(2)</td>
</tr>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>x</td>
<td>x (4)</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td></td>
<td>x (5)</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>x</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) Differential solution - equations of motion referenced to an element of mass, with the internal deformation modelled in detail.
(2) Integrated approach - depth-averaged shallow-flow solution, referenced to columns of debris mass above the sliding surface.
(3) Particulate modelling - discrete particle modelling.
(4) Smooth particle hydrodynamic approach is adopted.
(5) Particle-in-cell approach is adopted.
(6) Solution for motion of particles by distinct element method.
Table 3(b): Summary of method of analysis - solution dimensions

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>2-D(1)</th>
<th>Pseudo-3D(2)</th>
<th>3-D(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>x(4)</td>
<td>x(4)</td>
<td></td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>x(4)</td>
<td></td>
<td>x(4)</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td></td>
<td></td>
<td>x</td>
</tr>
</tbody>
</table>

Notes:
(1) Two-dimensional (2-D) - analysing cross-section of a single pre-defined width.
(2) Pseudo-three-dimensional (Pseudo-3D) - analysing cross-sections of varying pre-defined widths along debris trail to account for the plan dimension.
(3) Three-dimensional (3-D) - analysis in plan and in cross-section.
(4) Unlike the other models in the ‘2-D’ or ‘3-D’ category, TOCHNOG and PFC do not involve the use of depth-averaged shallow-flow solution. TOCHNOG and PFC simulate both the failure at the landslide source and the runout of the debris, without the need for imposing a pre-determined failure surface.
Table 3(c): Summary of method of analysis - solution reference frame

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Eulerian(1)</th>
<th>Lagrangian(2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>With mesh</td>
<td>Mesh Free</td>
</tr>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>N (4)</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>●</td>
<td>● (4)</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>RAMMS</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>DAN3D(NGI)</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>FLO-2D(NGI)</td>
<td></td>
<td>N</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>FLATMODEL</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>3dDMM</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>RASH3D(Paris)</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td>N or V</td>
<td>N</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>● (5)</td>
<td></td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>V (4)</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) Eulerian - fixed reference frame.
(2) Lagrangian - moving reference frame.
(3) For models that adopt an integrated approach, their reference column can be:
   Normal - depth-averaged in the direction of bed normal; denoted by “N”;
   Vertical - depth-averaged in the vertical direction; denoted by “V”.
(4) Differential approach with adaptive mesh technique.
(5) Distinct element method.
Table 3(d): Summary of method of analysis - basal rheology

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Frictional(1)</th>
<th>Voellmy(2)</th>
<th>Three-term(3)</th>
<th>Bingham(4)</th>
<th>Others(5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>x</td>
<td>x</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>x</td>
<td>x</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pouliquen friction</td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>Pouliquen friction</td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td>x</td>
<td>x</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td>Evolution function</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>Pouliquen friction</td>
<td></td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) Frictional - shear stress as linear function of total normal stress
(2) Voellmy - frictional term plus a term proportional to the square of velocity
(3) Three-term - frictional, viscous and turbulent
(4) Bingham - constant shear strength plus viscous term
(5) Others - e.g. Pouliquen friction
(6) Consideration of energy changes due to shear distortion of landslide debris is incorporated.
(7) Fully-coupled elastoplastic models for the landslide mass in a differential approach.
(8) Capable to model consolidation of landslide sliding surface.
(9) Rheological model is for inter-particle and particle-wall interaction in a distinct element approach.
(10) Variation of excess pore pressure in sliding surface is modelled.
Table 3(e): Summary of method of analysis - internal Stress and Energy Dissipation (other than basal) assumed in integrated models

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Hydrostatic&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>Rankine&lt;sup&gt;(2)&lt;/sup&gt;</th>
<th>“At Rest”&lt;sup&gt;(3)&lt;/sup&gt;</th>
<th>SH&lt;sup&gt;(4)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td></td>
<td>x (5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td></td>
<td></td>
<td></td>
<td>N.A.&lt;sup&gt;(6)&lt;/sup&gt;</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>x (7)</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td></td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td></td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>x (6)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td></td>
<td></td>
<td></td>
<td>N.A.&lt;sup&gt;(8)&lt;/sup&gt;</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td></td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td></td>
<td>x (9)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. Zero internal strength (i.e. hydrostatic, $k=1$).
2. Rankine stress state.
3. “At Rest” stress state (i.e. $k_a=k_p$).
4. Savage-Hutter (SH) model.
5. Consideration of energy changes due to shear distortion of landslide debris is incorporated.
6. Fully-coupled elasto-plastic soil models are available for calculation of internal stress distribution.
7. Based on formulation proposed by Bartelt et al. (1999).
9. Not certain based on information provided.
Table 3(f): Summary of method of analysis - entrainment of material from the path

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Not considered</th>
<th>User-specified&lt;sup&gt;(1)&lt;/sup&gt;</th>
<th>Algorithm-specified&lt;sup&gt;(2)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>x</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>RAMMS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>DAN3D(NGI)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>FLO-2D(NGI)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>RASH3D(Paris)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td></td>
<td></td>
<td>x</td>
</tr>
</tbody>
</table>

Notes:
(1) User-specified - entrainment rate or amount specified by the user.
(2) Algorithm-specified - rate and amount calculated by a pre-scribed algorithm, considering material properties.
Table 3(g): Summary of method of analysis - variation of basal strength along the path

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Not Considered</th>
<th>Considered</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td></td>
<td>x</td>
</tr>
</tbody>
</table>

Related Technical Papers
Apart from providing the modelling results on the benchmarking cases, all of the 13 teams have also submitted technical papers to describe their modelling methodologies and present the results of use of the models. The papers include:

(a) “Analysis of Hong Kong debris flow with an energy based model” by D Chan, N R Morgenstern, D Tran and X B Wang
(b) “Landslide mobility analysis using MADflow” by J H Chen and C F Lee
(c) “Approach to numerical modelling of long runout landslides” by G B Crosta, S Impossimo and D Roddeman
(d) “The 2005 Tate’s Cairn debris flow: Back-analysis, forward predictions and a sensitivity analysis” by J Cepeda
(e) “Application of 2D-finite volume code FLATModel to landslide runout benchmarking exercises” by M Hürlimann, V Medina and A Bateman
(f) “Benchmarking exercise on landslide mobility modelling - runout analyses using 3dDMM” by J S H Kwan and H W Sun
(g) “Benchmark exercises for granular flows” by A Lucas, A Mangeney, F Bouchut, M-O Briseaux and D Mége
(h) “Two models for analysis of landslide motion: Application to the 2007 Hong Kong benchmarking exercises” by O Hungr, M McKinnon and S McDougall
(i) “A SPH depth integrated model with pore pressure coupling for fast landslides and related phenomena” by M Pastor, T Blanc, M J Pastor, M Sánchez, B Haddad, P
RESULTS OF MODELLING

Group A Cases
The purpose of these cases is to serve as verification exercises. Verification serves to ensure that a given model produces verifiably accurate results for a range of geometries and simple, independently derived, material properties. Verification exercises usually rely on physical laboratory model cases, conducted under closely controlled conditions and with simple materials whose rheological behaviour is well-defined (such as dry sand). Two laboratory test cases and a dam-break scenario are included in this group for benchmarking the modelling results. Seven teams have attempted the dam-break scenario and the laboratory test of Deflected Sand Flow, while five teams attempted the USGS Flume Test.

Dam-break Scenario
This is a 2-D case with an analytical solution available for direct comparison with the simulation results and validating the debris runout and debris profile simulated by the numerical models at different times after the ‘dam-break’. In the analytical solution, the debris mass is assumed to be frictionless internally and resistance to flow is derived from basal friction only. Seven teams submitted simulation results for this case (see Table 4).
Table 4: Teams participating in modelling of the dam break scenario

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>The initial boundary condition adopted is different from the analytical solution; the assumption of energy loss due to internal shear distortion calculated by the model is not considered by the analytical solution</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>The model considers frictional material when calculating the internal stress, i.e. different from the frictionless material assumed by the analytical solution</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL*</td>
<td>Results submitted before the Forum were clarified and replacement figures were provided after the Forum. Observation presented is based on the resubmitted results</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>A special 2-D version is used</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D *</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RASH3D *</td>
<td>RASH3D was used for comparison purposes</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>A special 2-D version is used</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>A special 2-D version is used, “wet bed” simulation results are provided</td>
</tr>
</tbody>
</table>

Figure 1 depicts the simulation results produced by various models of the participants at 10 s, 20 s and 30 s after ‘dam break’.

Observations
(a) With the same basal rheology and material parameters specified in the exercise, the six models marked * in Table 4 above, viz. FLATMODEL, 3dDMM, SHALTOP-2D, RASH3D, DAN and Pastor, give simulation results that match well with the analytical solutions at 10 seconds, 20 seconds and 30 seconds.

Notes:
- SHALTOP-2D and FLATMODEL gives an excellent match at all points.
- At the distal end of the debris where the debris depth is within about 1 m, the 2-D version of DAN3D gives a simulated debris depth that is greater than the analytical solution.
- The 2-D version of 3dDMM tends to give a simulated debris depth that is slightly smaller than the analytical solution.
- The “wet bed” simulation results of Pastor match well with the analytical solution; the depth of “wet bed” is 1 m.
- The RASH3D results provided by the participant show some minor deviations from the analytical solution. It is not certain whether this is related to the grid/mesh size adopted in the modelling.

(b) A landslide mass with internal friction was adopted in TOCHNOG’s and Wang’s modelling. The modelling results are not comparable to the analytical solution, which assumes that the debris mass has no internal friction.
The source conditions and initial boundary conditions adopted in Wang’s modelling were different from the analytical solution. Direct comparison between the modelling results and the analytical solutions is not viable.

Figure 1: Simulation results by various models for the dam break scenario
Deflected Sand Flow

The deflected sand flow experiment was carried out by the Rock Mechanics Laboratory of the Swiss Federal Institute of Technology in Lausanne (EPFL). The experiment involves releasing dry fine sands from a box placed in a flume, which was set up using two inclined planes sloping at different angles. Seven teams submitted simulation results for this case (see Table 5).
Table 5: Teams participating in modelling of the deflected sand flow

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>2D model</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW *</td>
<td></td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td></td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D *</td>
<td>A different bed friction (35°) is adopted</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D *</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2 depicts the simulation results produced by the various models of the participants. The contour lines at deposition zones of the test results, together with simulation results corresponding to thickness of 0.03 m and 0.005 m respectively, are marked for easy comparison.

**Observations**

(a) Based on the use of relevant basal rheology and material parameters, all the six 3-D models marked * in Table 5 produce simulations that resemble the test in respect to the overall reach of the sand flow and the broad shape of sand deposition.

(b) At 0.03 m debris depth, MADFLOW, 3dDMM and DAN3D give a consistent maximum runout, which matches well with the test results. The maximum runout at 0.03 m debris depth given by Pastor and RASH3D is slightly less than that of the test results, while the maximum runout of TITAN2D is greater than that of the test results. In respect of the area bounded by the 0.03 m contour, the test results match better with the simulation results of MADFLOW, 3dDMM and DAN3D, than with the simulation results of Pastor, RASH3D and TITAN2D.

(c) At 0.005 m debris depth, MADFLOW produces the best match with the test results. While the results of DAN3D match well with the test results at the debris front, some debris is seen to be deposited along the trail which is not observed in the test. Pastor, 3dDMM and RASH3D give reasonable simulation results but they slightly over-estimate the extent of the sand deposition.

(d) It should be noted that a better match with the test results does not necessarily mean better performance of the model, given the possible variations in material properties and test conditions. However, judging from (b) and (c) above, it appears that MADFLOW, 3dDMM, DAN3D, RASH3D and Pastor produce overall trend results that are consistent with the test results. TITAN2D results in a much greater degree of spreading of the debris deposition, as compared with the results from the other models and from the test results. This can also be illustrated by a comparison of the maximum debris deposition depth as obtained by the various models, as summarised in Table 6.
Figure 2: Simulation results by various models for the deflected sand flow experiment
Table 6: Maximum debris depth estimated by various models

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Maximum Debris Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW *</td>
<td>0.110 – 0.120</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>0.090 – 0.100</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>0.080 – 0.090</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>0.130 – 0.150</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D *</td>
<td>0.075</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>0.040 – 0.050</td>
</tr>
</tbody>
</table>

Notes: The maximum debris depth measured in the test is 0.13 m.

(e) The variations in the maximum debris deposition depth and in the spatial extent and profile of the debris deposition zone among the simulation results, including those obtained by the models marked * in Table 6, are notable. This may be related to the different assumptions made of the internal stress and the different numerical approaches (e.g. SPH and PIC) adopted in the models. While the runout distance is usually less sensitive to these factors, they may affect the debris deposition profile and its spatial coverage in some circumstances.

USGS Flume Test
The USGS reported two dry sand flow experiments using a miniature flume (Iverson and Delinger, 2004). The two experiments, Experiments A and B, were conducted with flume bed of different topography. Five teams attempted this benchmarking case, four of which used 3-D numerical models for the simulation (see Table 7). Two of the four teams submitted simulation results for both Experiments A and B.

Table 7: List of teams participating in modelling the USGS flume test

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>2D model</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL *</td>
<td>Experiment A only</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>Both Exp. A and B attempted</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D *</td>
<td>Experiment A only</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>Both Exp. A and B attempted</td>
</tr>
</tbody>
</table>

Figure 3 depicts the simulation results produced by the various models.

Observations
(a) Based on the use of relevant basal rheology and material parameters, all the four 3-D
models marked * in Table 7 above give simulation results that resemble the experimental results in respect of debris profiles and the overall runout of the debris.

(b) The experimental results provide good information on the runout and deposition of the debris, as well as the profile of the material that remains at the source, corresponding to different simulation times (up to 8 seconds). The four 3-D models also provide clear information on these aspects, which facilitates a direct comparison. The simulation results of the FLATMODEL, 3dDMM and SHALTOP-D are very similar, and they generally match well with the experimental results in respect of the debris runout. These models are able to capture the overall behaviour of the sand flows and the results reflect the influence of the 3-D profiles of the test flumes. Pastor also appears to give simulation results that are similar to the experiment, but judging from the contour plots given by the participant, the resolution of the result is notably coarser than those given by FLATMODEL, 3dDMM and SHALTOP-2D. It is not certain whether this might have been affected by the DEM/boundary conditions adopted in Pastor’s modelling for this case.

Figure 3: Simulation results by various models for the USGS flume test
As observed in the simulation of the deflected sand flow, there are some variations in the extent and profile of the deposition zone produced by the 3-D models. This can be illustrated by a comparison of the maximum debris depth when time is at 8 seconds, as summarised in Table 8 below. 3dDMM under-estimates the deposition depth, as its results show a more dispersed deposition zone, with larger lateral spreading as compared with the others.
All four 3-D models over-estimate the amount of material detached from the source, i.e. less material remains in place at the source as compared with the experimental results. This is anticipated because these dynamic models do not model the geometry of the slope at limiting equilibrium condition at the source. SHALTOP-2D gives simulation results that differ noticeably from the experimental results at and near the source, showing sand deposition around the opening of the sand discharge. It is not clear whether this might have been affected by the assumptions adopted by SHALTOP-2D in simulating the conditions at and near the source area.

Table 8: Maximum simulated debris depth at 8 seconds

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Maximum Debris Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>0.026 – 0.028</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>0.008 – 0.013</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>~0.05</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>0.023 (2)</td>
</tr>
</tbody>
</table>

Notes:
(1) The maximum debris depth measured in Experiment A and Experiment B at t = 8 seconds are 0.023 m and 0.016 m respectively.
(2) Simulation result at t = 3.99 seconds; results at t = 8 seconds were not submitted.
(3) Simulation result at t = 1.11 seconds; results at t = 8 seconds were not submitted.

Group B and C Cases
A review of the simulation results of the calibration cases in Group A indicates that certain models perform consistently and produce similar modelling results when in direct comparison with each other, although some variations exist among their debris deposition depths and lateral spreading of the debris. The results of some other models appear to be less consistent, possibly because of the limitations of their formulations in performing 3-D modelling, particularly for complex 3-D ground profiles and the difficulty in realistically accounting for a wide range of material properties and landslide types.

The outcome of the modelling of the nine actual landslide cases in Groups B and C are summarised below.

Tate’s Cairn Landslide
Seven teams set up their dynamic models for simulation of the Tate’s Cairn landslide as summarised in Table 9. Five of the seven teams used Voellmy rheology in the simulations. FLATMODEL also considers turbulence in its simulations, but it calculates the turbulent friction based on the Chezy coefficient. (The square of the Chezy coefficient is equivalent to the Voellmy turbulence coefficient). Apart from assuming the Voellmy rheology, NGI also carried out a series of back-analyses using FLO-2D based on quadratic rheology. The UBC group presented sets of results in the form of a simple parametric study.
Table 9: Team participating in simulation of Tate’s Cairn landslide and parameters adopted

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Base Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>DAN3D(NGI) *</td>
<td>15</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td>FLO-2D</td>
<td>#</td>
<td>#</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>23</td>
<td>400 $^{(2)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>25</td>
<td>998.6 $^{(2)}$</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>15</td>
<td>500</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>5.7 ($f = 0.1$) $^{(1)}$</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.7 ($f = 0.3$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26.6 ($f = 0.5$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5.7 ($f = 0.1$) $^{(1)}$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.7 ($f = 0.3$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>26.6 ($f = 0.5$) $^{(1)}$</td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>16.7 ($f = 0.3$)</td>
<td>500</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>25</td>
<td>1000</td>
</tr>
</tbody>
</table>

Notes:
(1) The sets of input parameters, i.e. ($f = 0.1, \xi = 300$ m/s$^2$), ($f = 0.1, \xi = 500$ m/s$^2$) and ($f = 0.5, \xi = 500$ m/s$^2$), do not produce good simulation results in terms of debris reach as compared with field observations and therefore they are not taken into consideration in the result comparison.
(2) Calculated based on the reported Chezy coefficient.
# The team back-analysed the case using quadratic rheology. A series of back-analyses was preformed, the set of parameters that produce the best-fit results are in the range of yield strength of 9 to 10 Pa, viscosity of 3 to 11 Pa·s and Manning’s n-value of 0.04.

Figure 4 shows the debris flow paths and the debris runouts. All the simulations produce a good match with the observed runout distance. The simulated debris travelling times and maximum debris deposition depths are given in Table 10.
Figure 4: Simulation results by various models for Tate’s Cairn landslide
Figure 4 (Con’t): Simulation results by various models for Tate’s Cairn landslide

- **DAN3D**
  - $f = 0.1$, $\xi = 300 \text{ m}^2/\text{s}$
  - $f = 0.3$, $\xi = 500 \text{ m}^2/\text{s}$

- **Pastor**
  - Maximum depth = 0.8 to 1.0
  - $\phi = 25^\circ$, $\xi = 1000 \text{ m}^2/\text{s}$

- **RASH3D**

- **3dDMM**
  - $\phi = 25^\circ$, $\xi = 1000 \text{ m}^2/\text{s}$
Table 10: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Base Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s$^2$)</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>25</td>
<td>-</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>DAN3D(NGI)*</td>
<td>15</td>
<td>1000</td>
<td>90</td>
<td>2.5 – 3.0</td>
</tr>
<tr>
<td></td>
<td>FLO-2D</td>
<td>N.A.</td>
<td>N.A.</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Technical University</td>
<td>FLATMODEL</td>
<td>23</td>
<td>400</td>
<td>-</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td>of Catalonia</td>
<td></td>
<td>25</td>
<td>1000</td>
<td>-</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>15</td>
<td>500</td>
<td>40</td>
<td>2.5 – 3.0</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>16.7 (f = 0.3)</td>
<td>300</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>26.6 (f = 0.5)</td>
<td></td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16.7 (f = 0.3)</td>
<td>500</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>16.7 (f = 0.3)</td>
<td>500</td>
<td>51</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>25</td>
<td>1000</td>
<td>250</td>
<td>0.8 – 1.0</td>
</tr>
</tbody>
</table>

Observations
(a) A direct comparison of the overall reach of the debris and debris flow path among the four models marked * in Table 6 above, i.e. DAN3D(NGI), 3dDMM, DAN3D and Pastor, is possible as they use similar basal rheology and material parameters. The four models give comparable simulation results.

(b) There are some variations in the maximum debris deposition depth, as well as the extent and profile of the debris deposition zone among the simulation results, including those obtained by the 3D models marked *.

(c) Debris travelling time has not been reported by all teams, but for those that have reported, viz. 3dDMM and Pastor, their debris travelling times are reasonably close to each other.

(d) Although FLATMODEL and RASH3D have adopted almost the same set of material parameters, they give different simulation results. The extent of debris trail simulated by the FLATMODEL appears to be larger as compared with RASH3D. Based on the available information, it is not able to directly compare the simulation results of FLATMODEL and RASH3D with those of the other models in (a) above. In an effort to benchmark these two models, GEO has carried out a simulation using 3dDMM based on the same set of Voellmy parameters (i.e. $\phi = 25^\circ$ and $\xi = 1000$ m/s$^2$). The 3dDMM simulation predicts that the debris would take about 30 seconds to reach the bend at about CH 220 but the debris front almost stops at this location. This is very similar to the RASH3D’s results. In the simulation of RASH3D, debris takes another 220 seconds to reach the actual final deposition location. However, 3dDMM does not produce the same results as RASH3D; the final debris runout of 3dDMM is about 40 m less (see Figure 4).
(e) Since debris profiles at different simulation times calculated by FLATMODEL were not submitted, no comparisons between FLATMODEL and 3dDMM can be made as above.

(f) The debris travelling time and maximum debris deposition thickness produced by FLO-2D were not reported; hence a comparison with others in these aspects cannot be made. Nevertheless, attention should be drawn to the fact that FLO-2D requires quite different inputs from the others in particular; it requires an inflow hydrograph at the landslide source area whereas an initial thickness of the landslide mass is specified for the other models. In the case of Tate’s Cairn landslide, the inflow hydrograph is not available. To use FLO-2D to simulate this landslide case, NGI devised an inflow hydrograph based on DAN3D’s results.

**Tate’s Cairn Landslide Forward Prediction**

The following six sets of model parameters were provided to the participants for analysing the debris runout of a 10,000 m$^3$ landslide in the forward prediction exercise (see Table 11):

<table>
<thead>
<tr>
<th>Case</th>
<th>Debris Entrainment Ratio</th>
<th>Voellmy Model Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Apparent Friction Angle ($^\circ$)</td>
</tr>
<tr>
<td>1(a)</td>
<td>100%</td>
<td>8$^\circ$</td>
</tr>
<tr>
<td>1(b)</td>
<td>0%</td>
<td>8$^\circ$</td>
</tr>
<tr>
<td>2(a)</td>
<td>100%</td>
<td>15$^\circ$</td>
</tr>
<tr>
<td>2(b)</td>
<td>0%</td>
<td>15$^\circ$</td>
</tr>
<tr>
<td>3(a)</td>
<td>100%</td>
<td>25$^\circ$</td>
</tr>
<tr>
<td>3(b)</td>
<td>0%</td>
<td>25$^\circ$</td>
</tr>
</tbody>
</table>

Not all the teams have attempted and presented all six cases. Table 12 summarises the modelling results received.
Table 12: Teams participating in modelling of Tate’s Cairn landslide forward prediction

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>1(a)</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>1(b)</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>2(a)</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>DAN3D (NGI) *</td>
<td>3(a)</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor (3)</td>
<td>3(b)</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. • represents cases attempted and results presented.
2. Only graphical outputs of erosion depth for Cases 1(a) and 3(a) are reported. No indication of deposition depth is presented in the graphical outputs, therefore a direct comparison with the other modelling results in terms of debris runout and debris flow path is not possible. Case 2(a) was reportedly attempted but results have not been given by the team.
3. The team has adopted an initial landslide volume of 11052 m$^3$, $\phi = 16.7^\circ$ (i.e. $f = 0.3$) and $\xi = 500$ m/s$^2$ in the forward prediction exercise, the results of which cannot be compared with those of the others that follow the given initial landslide volume and model parameters.
4. Case 2(b) was reportedly attempted but the results have not been presented by the team.

Figures 5(a) and 5(b) show the debris flow paths and debris runout for (a)-series and (b)-series cases respectively. Tables 13(a) and 13(b) summarise the debris travelling times and maximum debris deposition depths of the (a)-series and (b)-series cases respectively.

Table 13(a): Summary of debris travelling times and maximum debris deposition depths in (a)-series

<table>
<thead>
<tr>
<th>Case</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(a)</td>
<td>3dDMM</td>
<td>40</td>
<td>2.0 – 2.5</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>2.0 – 3.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>-</td>
<td>6.1 – 8.0 ^</td>
</tr>
<tr>
<td>2(a)</td>
<td>3dDMM</td>
<td>60</td>
<td>4.5 – 5.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>3.0 – 4.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>-</td>
<td>4.1 – 6.0 ^</td>
</tr>
<tr>
<td>3(a)</td>
<td>3dDMM</td>
<td>50</td>
<td>3.5 – 4.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>1.0 – 2.0</td>
</tr>
</tbody>
</table>

Notes:
^ denotes the maximum debris depth (listed in the table for reference) but not the maximum deposition depth.
### Table 13(b): Summary of debris travelling times and maximum debris deposition depths in (b)-series

<table>
<thead>
<tr>
<th>Case</th>
<th>Model</th>
<th>Debris Travelling Time</th>
<th>Maximum Debris Deposition Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(s)</td>
<td>(m)</td>
</tr>
<tr>
<td>1(b)</td>
<td>FLATMODEL</td>
<td>-</td>
<td>11.6 ^</td>
</tr>
<tr>
<td></td>
<td>3dDMM</td>
<td>50</td>
<td>2.0 – 2.5</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>2.0 – 3.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>-</td>
<td>7.6 ^</td>
</tr>
<tr>
<td></td>
<td>RASH3D</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>2(b)</td>
<td>FLATMODEL</td>
<td>-</td>
<td>8.2 ^</td>
</tr>
<tr>
<td></td>
<td>3dDMM</td>
<td>60</td>
<td>3.5 – 4.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>2.0 – 3.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>-</td>
<td>7.6 ^</td>
</tr>
<tr>
<td>3(b)</td>
<td>FLATMODEL</td>
<td>-</td>
<td>5.5 ^</td>
</tr>
<tr>
<td></td>
<td>3dDMM</td>
<td>50</td>
<td>3.5 – 4.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D</td>
<td>-</td>
<td>1.0 – 2.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>-</td>
<td>~5 ^</td>
</tr>
<tr>
<td></td>
<td>RASH3D</td>
<td>-</td>
<td>3.2</td>
</tr>
</tbody>
</table>

**Notes:**

^ denotes the maximum debris depth (listed in the table for reference) but not the maximum deposition depth.
Figure 5(a): Simulation results by various models for Tate’s Cairn landslide forward prediction
Figure 5(a) (Con’t): Simulation results by various models for Tate’s Cairn landslide forward prediction

Figure 5(b): Simulation results by various models for Tate’s Cairn landslide forward prediction
Figure 5(b) (Con’t): Simulation results by various models for Tate’s Cairn landslide forward prediction
Case 2 (b)

Approximate location of houses

Figure 5(b) (Con’t): Simulation results by various models for Tate’s Cairn landslide forward prediction
Observations
(a) The modelling results of Cases 1(a) and 1(b) cannot be compared directly, as the frontal portion of the landslide, in all the models, has run out of the extent of the given modelling boundary. The predicted distal ends of the landslide have not been reported.

(b) Based on the modelling results of Cases 2(a), 2(b), 3(a) and 3(b) in terms of the overall runout of the debris and debris flow path, the three models marked * in Table 12 above, i.e. DMM, DAN3D and DAN3D(NGI), give similar simulation results, though there are some discrepancies in the maximum debris deposition depth among them.

(c) Only the 3dDMM team reports the debris travelling time, no other teams have reported on this. It is not therefore possible to provide comments on the debris runout time.

(d) The modelling results of FLATMODEL for Cases 2(b) and 3(b) indicate a longer runout distance as compared with those by 3dDMM, DAN3D and DAN3D(NGI).

Fei Tsui Road Landslide
Eight teams set up their dynamic models for simulation of Fei Tsui Road landslide. Six of the eight teams used pure frictional model in their simulations. The values of basal friction angle adopted by all the participants are very similar.

Table 14 summaries the teams and the model parameters adopted for simulation of Fei Tsui Road Landslide.

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ ($^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Internal: 30&lt;br&gt;Basal: 30</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW #</td>
<td>Internal: 35&lt;br&gt;Basal: 22&lt;br&gt;</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>Internal: 35&lt;br&gt;Basal (at landslide scar): 22&lt;br&gt;Basal (elsewhere): 35</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D #</td>
<td>26</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>Basal (on slope): 20&lt;br&gt;Basal (elsewhere): 35</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor #</td>
<td>26.6</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>21.8 (1)</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D #</td>
<td>27</td>
</tr>
</tbody>
</table>

Notes:
(1) This is the initial basal friction. Since changes in the pore water pressure are considered by the model, the apparent friction angle at the base is subject to change during the course of simulation.

Figure 6 shows the debris flow paths and debris runout of the modelling results. A summary of the debris travelling times and maximum debris deposition depths are given in Table 15.
Figure 6: Simulation results by various models for Fei Tsui Road landslide
### Table 15: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Overall</td>
<td>At Church</td>
</tr>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>14</td>
<td>9</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>20</td>
<td>5.5</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>13</td>
<td>-</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>15</td>
<td>8</td>
</tr>
</tbody>
</table>

**Observations**

(a) A direct comparison of the overall runout of the debris and debris flow path among the two models marked * in Table 14 above, i.e. 3dDMM and DAN3D, is possible as they use similar parameters. The two models perform similarly and give comparable simulation results, although DAN3D over-estimates the final deposition area within a thin debris thickness < 0.5 m.

(b) Similarly, the four models marked # in Table 14 above, i.e. MADFLOW, SHALTOP-2D, Pastor and RASH3D, give simulation results that match well each other using similar basal rheology and parameters.

(c) A further examination of the models in (a) and (b) above indicates that their modelling results, in terms of the overall runout of the debris and debris flow path, resemble each other reasonably. Furthermore, the maximum debris deposition depths obtained by all the models do not deviate very much from each other, although some variations exist as are also noted in other benchmarking cases.

(d) It is noted that the rheological parameters adopted by Sassa-Wang’s model is different from those adopted in other models. Sassa-Wang’s model considers changes in the apparent friction angle due to consolidation. Apart from noting that the initial friction angle used by Sassa-Wang (i.e. 21.8°) is lower than that by the others, further details on Sassa-Wang’s model are not available for a more in-depth review.

**Shum Wan Landslide**

Results of two 2-D models and seven 3-D models of this landslide case were submitted by participants. One of the participating teams, University of Milano Bicocca and FEAT, submitted both 2-D and 3-D modelling results produced by the model TOCHNOG. The list of participants and the model parameters used are presented in Table 16.
Table 16: List of participants and the input parameters for Shum Wan landslide

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Internal: 30 Basal: 16</td>
<td>-</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW *</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>University of Milano, Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>2-D: 22 – 26 3-D: 20 – 22</td>
<td>-</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>Internal: 38 Basal: 40 (shipyard) 15 (elsewhere)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal: 38 Basal: 40 (shipyard) 15 (elsewhere)</td>
<td>500</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D *</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>22</td>
<td>11.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>200</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>16.7</td>
<td>1000</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>21.8 (1)</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
(1) This is the initial basal friction. Since changes in the pore water pressure are considered in the model, the apparent friction angle at the base is subject to change during the course of simulation.

Figure 7 shows the debris flow paths and the final debris deposition profiles of the different models. Table 17 summarises the debris travelling times and maximum debris deposition depths.

Table 17: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>20</td>
<td>~9</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>30</td>
<td>5.0</td>
</tr>
<tr>
<td>University of Milano, Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>2-D: 20 3-D: 50</td>
<td>2-D: ~9 3-D: 11.0</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>DMM</td>
<td>Frictional: 28 Voellmy: 24</td>
<td>7.0 – 8.0</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>28</td>
<td>7.0 – 8.0</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>-</td>
<td>5.0 – 6.0</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>29</td>
<td>8.0</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>26</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 7: Simulation results by various models for Shum Wan landslide
Figure 7 (Con’t): Simulation results by various models for Shum Wan landslide

**Observations**

(a) Figure 7 indicates that the four models marked * in Table 16, i.e. MADFLOW, 3dDMM, DAN3D and SHALTOP-2D, give similar modelling results in terms of the overall runout of the debris and the broad shape of debris deposition. The debris flow paths and the maximum debris deposition depth are also comparable with each other.

(b) The simulation results of TOCHNOG in terms of the overall runout of the debris and the broad shape of debris deposition resemble those produced by the 3-D models in (a). Nevertheless, the debris travelling time and the maximum debris deposition depth are slightly greater than those of the models in (a). Little information on the exact debris travelling time/velocities is available in this landslide case, and hence it cannot be further assessed which of the models have simulated a more accurate travelling time.

(c) The simulation results produced by the 2-D models TOCHNOG and Wang are very similar in terms of the debris travelling time, the maximum deposition depth and runout distance. However, direct comparison of the results of the two models is not possible, since the two models adopt very different calculations of internal stress and energy dissipation. In addition, TOCHNOG simulates the conditions of failure of the groundmass without using the depth-averaged shallow-flow assumption, which is by nature different from the method adopted by Wang and other integrated approach models.

(d) The overall runout of the debris simulated by Sassa-Wang’s model appears to be comparable with the models in (a) and (b) above. However, since details such as the final deposition depth are not reported, further comparisons on other aspects are not possible.

(e) GEO has also simulated the landslide with 3dDMM using a frictional basal rheology with $\phi = 20^\circ$ apart from those reported before the Forum. The results are given in the paper submitted separately for publication in the Forum Proceedings. The simulated deposition profile of this additional analysis is also shown in Figure 7. With this basal rheology, the modelling results in terms of the overall runout of the debris and the broad...
shape of debris deposition of 3dDMM can be compared directly with those of MADFLOW, DAN3D and SHALTOP-2D, which have adopted similar basal rheology and material parameters.

**Frank Slide**

Nine teams submitted their simulation results of the Frank Slide. Seven of them used frictional rheology in their simulations, while the other two teams used Voellmy rheology. Table 18 shows the participants and the parameters adopted.

Table 18: List of participants and the input parameters for Frank slide

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle</th>
<th>Turbulent Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>Internal: 25</td>
<td>Basal: 12 (source area) Basal: 16 (other areas) -</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>Internal: 40</td>
<td>Basal: 12 -</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D *</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D #</td>
<td>Basal: 15° (source area) Basal: 5.7° (f = 0.1) (path) Source Area: N/A Path: 500</td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>12.4 (f = 0.22)</td>
<td>-</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>36.9 (f = 0.75)</td>
<td>-</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D #</td>
<td>5.7</td>
<td>700</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D *</td>
<td>14</td>
<td>-</td>
</tr>
</tbody>
</table>

The debris flow paths and debris runout of the different models are presented in Figure 8. Table 19 summarises the debris travelling times and maximum debris deposition depths given by the models.

Table 19: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>90</td>
<td>25 – 30</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>60</td>
<td>15 – 20</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>100</td>
<td>45 – 50</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>-</td>
<td>20 – 25</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>71</td>
<td>25 – 30</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>80</td>
<td>-</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa-Wang</td>
<td>120</td>
<td>-</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>60</td>
<td>25 - 30</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>55</td>
<td>20 - 25</td>
</tr>
</tbody>
</table>
Figure 8: Simulation results by various models for Frank Slide
Observations

(a) For models that adopt frictional basal rheology, the models marked * in Table 18, i.e. 3dDMM, SHALTOP-2D, Pastor and TITAN2D, give modelling results that resemble each other in terms of the overall runout of the debris and the broad shape of debris deposition.

(b) Similarly, for models that adopt Voellmy basal rheology, the models marked # in Table 18, viz. DAN3D and RASH3D, give comparable modelling results in terms of the overall runout of the debris and the broad shape of debris deposition.

(c) The modelling results of TOCHNOG (which adopts an elasto-plastic rheology for the landslide mass elements) appear to match with those of the models in (a) above. (Note: The outline of the simulated debris deposition zone given in Fig. 5 of the paper by the TOCHNOG team submitted to the Forum Proceedings also shows a good match with the actual landslide.)

(d) The modelling results in the form of cluster of particles produced by PFC enable only a comparison of the extent of the debris deposition with the other models, as the rheological and energy dissipation models adopted are different from those adopted in other models. The overall runout of the debris as well as debris travelling time resembles that of the other models. No comment can be made for maximum debris deposition depth as related information has not been provided in the PFC submission.

(e) A review of the modelling results of Sassa-Wang’s model is difficult, as the submission does not contain sufficient details for comparison.

Sham Tseng San Tsuen Debris Flow

Five teams attempted this case. NGI used two 3-D models, DAN3D and RAMMS, to carry out simulations of the debris flow. They tried two sets of Voellmy parameters when using RAMMS, and they also set up DAN3D models using frictional and Voellmy rheologies. Participants who attempted the case and the model parameters used are summarised in Table 20.
Table 20: List of participants and the input parameters for Sham Tsang San Tsuen debris flow

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Internal: 35 Basal: 20</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>RAMMS – V (1) #</td>
<td>14</td>
<td>450</td>
</tr>
<tr>
<td></td>
<td>RAMMS – V (2)</td>
<td>Basal: 8.2 (exposed rock) 0.6 (waterfall) 31 (houses &amp; nullah) 14 (elsewhere)</td>
<td>450</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>DAN3D(NGI) – V #</td>
<td>14</td>
<td>450</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI) – F *</td>
<td>Internal: 35 Basal: 19.3</td>
<td>-</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>Internal: 30 Basal: 12</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>DAN3D – F *</td>
<td>17</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>DAN3D – V #</td>
<td>16.7 (f = 0.3)</td>
<td>500</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D – F</td>
<td>0.5 – 1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D – V</td>
<td>1.0 – 1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D – V</td>
<td>1.0 – 1.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D – F</td>
<td>0.5 – 1.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td>DAN3D – V</td>
<td>1.0 – 1.5</td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>20</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 9 shows the debris flow paths and debris reaches submitted by the participants. Table 21 presents the debris travelling times and maximum debris deposition depths.

Table 21: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>30</td>
<td>~7</td>
</tr>
<tr>
<td></td>
<td>RAMMS – V (1)</td>
<td>-</td>
<td>0.5 – 1.0</td>
</tr>
<tr>
<td></td>
<td>RAMMS – V (2)</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>DAN3D(NGI) – V</td>
<td>-</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI) – F</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>50</td>
<td>1.5 – 2.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D – F</td>
<td>-</td>
<td>0.5 – 1.0</td>
</tr>
<tr>
<td></td>
<td>DAN3D – V</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 9: Simulation results by various models for Sham Tseng San Tsuen debris flow
Observations
(a) The friction rheological parameters adopted in the models marked * in Table 20, viz. DAN3D(NGI), DAN3D and Pastor, are in similar order. This facilitates direct comparison among their modelling results. Figure 9 indicates that their modelling results are comparable with each other in terms of the overall runout of the debris and the broad shape of debris deposition.

(b) The Voellmy rheological parameters adopted in the models marked # in Table 20, viz. RAMMS, DAN3D(NGI) and DAN3D, are in similar order. A comparison of their modelling results based on Figure 9 shows that the models produce similar results, which have consistence in terms of the overall runout of the debris and the broad shape of debris deposition.

(c) In terms of the geometry of debris deposition, the simulated results of Wang’s model are notably different from that of the other models in the group. This may be related to the fact that Wang’s 2-D continuum model does not allow for separation of debris, whereas the other 3-D models permit this.

1990 Tsing Shan Debris Flow
Amongst the five teams that produced simulations of the 1990 Tsing Shan debris flow, four teams adopted Voellmy rheology, while Wang’s model used pure frictional rheology with explicit consideration of energy loss due to internal shear distortion. Table 22 summarises the participants and the parameters used in the models.
Table 22: List of participants and the input parameters for 1990 Tsing Shan debris flow

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ ($^\circ$)</th>
<th>Turbulent Coefficient $\zeta$ (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Internal: 35 Basal: 24</td>
<td>-</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>11.3</td>
<td>64 $^{(1)}$</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>Internal: 30 Basal: 15</td>
<td>500</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>11.3</td>
<td>500</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>10.2</td>
<td>500</td>
</tr>
</tbody>
</table>

Notes:
(1) This is calculated based on the reported Chezy coefficient of 8 m$^{1/2}$/s. The square of the Chezy coefficient is equivalent to the Voellmy coefficient.

Figure 10 presents the debris flow paths and debris runout given by the above models. Table 23 summarises the debris travelling times and maximum debris deposition depths.
Figure 10: Simulation results by various models for 1990 Tsing Shan debris flow
Table 23: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>-</td>
<td>4.0 – 4.5</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>50</td>
<td>2.5 – 3.0</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>-</td>
<td>2.0 – 3.0</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>81</td>
<td>-</td>
</tr>
</tbody>
</table>

Observations
(a) This case involves the simulation of entrainment effect along the debris flow path in order to accumulate the landslide source volume to its final landslide volume at deposition.
(b) It is noted that the approaches adopted by the different teams in simulating the entrainment effect are dissimilar with each other and therefore, there are notable differences among the modelling results submitted. These make direct comparison among them difficult. However, it is evident that the different assumptions made on the mode of debris entrainment in the models affected the simulation results.

2000 Tsing Shan Debris Flow
Three teams used 3-D model to simulate the 2000 Tsing Shan debris flow, which bifurcated at the top of a ridge line. The 2-D model of Wang was used for simulation of the debris flow, for which the participant set up two dynamic models to simulate the runout behaviours of each of the branches. A list of participants and parameters used are given in Table 24.

Table 24: Input parameters by different teams for 2000 Tsing Shan debris flow

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>14</td>
<td>-</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>8.53</td>
<td>400 (1)</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>Pylon Area: 45 Other Areas: 11</td>
<td>500</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>10.2 (f = 0.18)</td>
<td>500</td>
</tr>
</tbody>
</table>

Notes:
(1) This is calculated based on the report Chezy coefficient of 20 m$^{1/2}$/s.

Figure 11 shows the debris flow paths and debris runout. The debris travelling times and the maximum debris deposition depths estimated by the different model are given in Table 25.
Figure 11: Simulation results by various models for 2000 Tsing Shan debris flow
Table 25: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>40</td>
<td>-</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>-</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>25</td>
<td>1.0 – 1.5</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>81</td>
<td>1.5 – 2.0</td>
</tr>
</tbody>
</table>

**Observations**

(a) The 2000 Tsing Shan debris flow is included to examine the capability of the models in simulating debris flows over complex topography, including bifurcation of debris trail.

(b) Notwithstanding the basal rheology and material parameters adopted, the models marked * in Table 24 above, i.e. FLATMODEL, 3dDMM and Pastor, have successfully simulated the bifurcated debris trails.

(c) 3dDMM and Pastor give reasonable modelling results of the overall runout of the debris. The modelling results of FLATMODEL show a larger final deposition area, which is not observed by the other models.

**Thurwieser Rock Avalanche**

Five continuum models and one discrete particle model were used to set up the simulation model for this case. To reflect the varying frictional characteristics along the debris trail, which comprises glacier, together with steep and hummocky rocky terrain, all the continuum models apply different frictional parameters for different regions in the calculation domain. Table 26 presents a list of participants and the parameters used.
Table 26: List of participants and the input parameters for Thurwieser rock avalanche

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle $\phi$ (°)</th>
<th>Turbulent Coefficient $\xi$ (m/s$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>Region 1: 10$^\text{(1)}$</td>
<td>500</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>Region 2: 20</td>
<td>-</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM *</td>
<td>Glacial Area: 12</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Other Areas: 27</td>
<td>-</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D *</td>
<td>Source Area: 25 - 28</td>
<td>Source Area: N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Glacial Area: 5.7 (f=0.1)</td>
<td>Glacial Area: 1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Below Glacial Area: 28</td>
<td>Other Area: N/A</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor *</td>
<td>Glacial Area: 0</td>
<td>Glacial Area: 1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Other Areas: 21.3 (f = 0.39)</td>
<td>Other Areas: N/A</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes:
(1) Region 1 is above elevation 2600 m a.s.l. where rock outcrop exists.

The graphical results of the different models are presented in Figure 12. Table 27 summarizes the results.

![MADFLOW](image1)

![TOCHNOG](image2)

![Pastor](image3)

![PFC](image4)

Figure 12: Simulation results by various models for Thurwieser rock avalanche
Figure 12 (Con’t): Simulation results by various models for Thurwieser rock avalanche

Table 27: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>300</td>
<td>15 – 20</td>
</tr>
<tr>
<td>University of Milano Bicocca and FEAT</td>
<td>TOCHNOG</td>
<td>100</td>
<td>15 – 20</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>DMM</td>
<td>100</td>
<td>15 – 20</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN3D</td>
<td>-</td>
<td>20 – 25</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>684</td>
<td>30 – 35</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>500</td>
<td>-</td>
</tr>
</tbody>
</table>

Observations
(a) This is a complicated back-analysis case in terms of modelling different types of ground materials over which the debris travels and their related basal rheology, and developing a digital elevation model that has sufficiently good resolution to reflect reasonably the changes in the complex topography.

(b) Because of (a) above, the models have, based on their respective capability in 3-D modelling, adopted different sets of material properties and basal rheology to simulate the landslide. Thus, a direct comparison of their modelling results is difficult.

(c) Notwithstanding (a) and (b) above, it is noted that, based on the orders of the adopted input parameters, the models marked * in Table 26, i.e. 3dDMM, DAN3D and Pastor, have produced consistent modelling results in terms of the overall runout of the debris and debris flow path. In particular, branches of the debris at the source and at the toe are observed in their modelling results, indicating their capabilities in modelling debris flow over complex terrain. The simulations results also match well with the actual landslide.

(d) The modelling result of MADFLOW does not exhibit debris branches along the debris flow paths. This may possibly be due to the fact that the model has no provisions for splitting up connectivity among debris elements in their formulations.
(e) PFC also produces modelling results that resemble the landslide. It gives the debris branches of debris at the source.

**Lo Wai Debris Flood**

One set of 2-D and one set of 3-D simulation results of Lo Wai debris flood were submitted. Table 28 summaries the participants and parameters used. The results of the debris flood travelling times and maximum deposition depths are presented in Table 29.

Table 28: List of participants and the input parameters used for Lo Wai debris flood

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Friction Angle ( \phi ) (°)</th>
<th>Turbulent Coefficient ( \xi ) (m/s²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>Internal: 30&lt;br&gt;Basal: 9</td>
<td>-</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>(1)</td>
<td>(1)</td>
</tr>
</tbody>
</table>

Notes:
1. Pastor used a combined rheological model involving Bingham and Manning rheology. In general, the basal friction adopted by the Pastor’s model is the lesser of the Bingham and the Manning frictions.

Table 29: Summary of debris travelling times and maximum debris deposition depths

<table>
<thead>
<tr>
<th>Team</th>
<th>Model</th>
<th>Debris Travelling Time (s)</th>
<th>Maximum Debris Deposition Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>40</td>
<td>~4</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>Pastor</td>
<td>200 – 300</td>
<td>~3</td>
</tr>
</tbody>
</table>

**Observations**

On the basis that both of the above models adopted different rheologies, a direct comparison between the two models is not possible. Apart from the rheological models, the initial conditions specified in the models are probably different, as Wang’s model requires the input of a thickness of landslide mass at the source rather than inflow hydrograph, as it should be for this case involving overflow from a catchwater. These could be the reasons for the remarkable difference between the debris flow travelling times estimated by the two models. The time given by Wang’s simulation is one order of magnitude less than that of Pastor’s model.

**SUMMARY OF BENCHMARKING RESULTS**

(a) Four 3-D models that allow for frictional and Voellmy basal rheologies, viz. DAN3D, 3dDMM, Pastor and RASH3d, have consistently provided similar modelling results for a range of cases in the benchmarking exercise (Table 30). In terms of the debris runout path, travel distance, time and overall shape of the debris deposition, the simulation results from these models match reasonably well with the analytical/experimental results of the calibration cases as well as the field behaviour of the actual landslide cases. Similarly, in a number of cases where the frictional
rheology is adopted, SHALTOP-2D and MADFLOW also give results that are consistent with the four models. This is promising in view of the different 3-D numerical solution methodologies adopted by the models.

(b) TOCHNOG, RAMMS and TITAN2D provided modelling results in this benchmarking exercise on a relatively smaller number of cases. The modelling results of RAMMS are similar to those of the models described in (a) above (Table 30). The results of TOCHNOG and TITAN2D are also comparable, although there appear to be some discrepancies when compared with the simulation results of the other models. However, since the simulation results of TOCHNOG, RAMMS and TITAN2D for more cases in this benchmarking exercise are not available, it is not possible to further compare their results with those of other models in this exercise.

(c) Despite the apparently good fit of the simulation results from the models discussed in (a) and (b) above on the debris runout path, travel distance, time and overall shape of debris deposition, there are some notable discrepancies in the spatial extent and profile of the debris deposition zones as simulated by the models. The discrepancies may be related to the assumptions made in respect of the internal stress within the debris and the different approaches for 3-D numerical solution adopted by the different models.

(d) In evaluating the benchmarking results, apart from examining the debris runout paths and deposition zones, consideration should also be given to whether the simulated debris runout durations and velocities match with those of the actual cases. This has been considered as far as information is available from the benchmarking results provided by the participants and from the actual verification and landslide cases. However, since the participants have not been requested to extract the modelling results at specific time intervals of the simulated debris runout, direct comparison of the temporal distributions of the simulated reach and velocity of the debris among the participants, and with the actual verification and landslide cases, is difficult. This is an area for improvement in future benchmarking exercises.

(e) The simulation results of FLATMODEL and Sassa-Wang also match the debris runout and the broad extent of deposition zone, although some differences to the results of the other models are noted. For example, the deposition extents estimated by FLATMODEL appear to be larger as compared with the others for cases such as the Tate’s Cairn debris flow. On the other hand, a complete benchmarking of Sassa-Wang model with others cannot be made since details such as the final deposition depth of the models are not reported.

(f) Many of the models have previously been tested or calibrated with other cases and the results published separately. These results have not been further reviewed in this benchmarking exercise, and the observations made in this report are based on the modelling results on the benchmark cases submitted to this exercise.

(g) Submissions were received from the use of three other models, which are by nature quite different to the other 3-D models described above:
- PFC – The simulation results are available for a small number of cases. The simulated debris runout path and the overall shape of debris deposition resemble those of the actual landslides. Direct comparison with other models cannot be made due to the different rheological and energy dissipation assumptions adopted.
- FLO-2D – This was used for the Tate’s Cairn debris flow. The participants have not reported the use of other models for the simulation of the other landslide cases.

- Wang (i.e. University of Alberta) – This 2-D model includes consideration of energy changes associated with shear distortion of the debris. A direct comparison with the simulation results by the other 3-D models, the experimental results and the actual landslides is difficult given the 2-D nature of Wang’s model. In addition, the different boundary/source conditions assumed in Wang’s model have affected the results in some cases. Where comparison has been made, it suggests that Wang’s simulation results appear to be different from those of the other models with similar basal rheological parameters. This reflects possible differences of Wang’s model from the other models, including its consideration/assumption of energy dissipation due to shear distortion of the debris.

(h) In general, it is noteworthy that the use of a 3-D model has distinct advantages in simulating the source, runout flow path and deposition zone, which are 3-D in geometry. A number of numerical solution techniques can be applied, and these appear to be giving consistent results as observed in this Benchmarking Exercise. Some models allow for separation and merging of debris along the runout path, which is required in dealing with more complicated cases. In comparison, the allowance for entrainment is generally less well developed. The modes of entrainment assumed in the modelling can greatly affect the simulation results. Some models (e.g. Pastor, DAN3D, 3dDMM and FLATMODEL) allow for entrainment effects based on entrainment rates prescribed by users or empirical rules, while TECHNOG and PFC simulates erosion and deposition based on the material properties and the type of interaction with the topographic surface and material along the runout path.

(i) None of the models allows for changes in the terrain profile as a result of debris deposition and entrainment during the simulation, which may in some cases affect the debris flow path and the modelling results.

(j) By nature, models that are based on depth-averaged shallow-flow solution are not suitable for use in simulating landslide with very steep failure surface or where the debris thickness is large in comparison with the runout distance. In practice, it is normally not a problem in modelling mobile debris flows and long runout landslides.

REFERENCES


APPENDICES
Appendix A – Landslide Runout Analysis Benchmarking Exercise: Terms of Reference
Table 32: Summary of modelling results

<table>
<thead>
<tr>
<th>Team</th>
<th>Model Denoted As</th>
<th>Group A</th>
<th>Group B</th>
<th>Group C</th>
</tr>
</thead>
<tbody>
<tr>
<td>University of Alberta</td>
<td>Wang</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>University of Hong Kong</td>
<td>MADFLOW</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>University of Milan</td>
<td>TOCHNOG</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>NGI, Norway</td>
<td>RAMMS</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>DAN3D(NGI)</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>FLO-2D(NGI)</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Technical University of Catalonia</td>
<td>FLATMODEL</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>GEO, Hong Kong</td>
<td>3dDMM</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Universite Paris Diderot</td>
<td>SHALTOP-2D</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>RASH3D(Paris)</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>UBC, Vancouver</td>
<td>DAN</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>CEDEX, Madrid</td>
<td>DAN3D</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td></td>
<td>Pastor</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Vienna University of Technology</td>
<td>PFC</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Kyoto University</td>
<td>Sassa</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>Politecnico Di Torino</td>
<td>RASH3D</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
<tr>
<td>University at Buffalo, New York</td>
<td>TITAN2D</td>
<td>●</td>
<td>●</td>
<td>●</td>
</tr>
</tbody>
</table>

Note: [Yellow] represents similar modelling results from the models when similar rheology models and parameters are adopted.
APPENDIX A
LANDSLIDE RUNOUT ANALYSIS BENCHMARKING EXERCISE: TERMS OF REFERENCE

1. Introduction

Prediction of landslide motion distance and velocity is required for hazard and risk assessment and for design of risk mitigation measures. The goal of such predictions is to estimate the area that may be affected by the movement of a potential landslide and to map hazard intensity parameters, esp. velocity, depth of flow and thickness of deposits.

The technology for making such predictions has advanced substantially in recent years. Numerical computer-based models now exist, capable of simulating the motion of a given volume of unstable material from its source on a hillside to a deposition area. As the technology begins to mature, it is useful to compare the various models with one another. The Organizing Committee of the 2007 International Forum on Landslide Disaster Management, Hong Kong have decided to devote a one-day session at the Forum to a review of the current state-of-the-art of landslide dynamics modelling. The main subject of the session will be the presentation of the results of a Benchmarking Exercise, as described below. The session programme for this subject will include a theme lecture, selected presentations from the participants of the Benchmarking Exercise, a summary of the results by the Review Committee, and a round-table discussion.

The Benchmarking Exercise is not a competition! Its purpose is to assess whether this emerging field of science is on its way towards establishing some degree of commonality amongst different methods used by various groups. It is felt that forward runout predictions can only be regarded as sufficiently reliable once there is certain convergence of modelling philosophies and methodologies amongst various groups specializing in the subject.

2. The Benchmarking Exercise

2.1 Data to be supplied to Participants

The organizers have selected nine examples of landslides and one laboratory test involving the flow of dry sand. A dam-break scenario, for which analytical solutions exist, is also included in the Benchmarking Exercise. All the cases involve long-runout landslides or mobile debris floods, moving at extremely rapid velocities. The exercise does not include slow-moving slides such as earth flows, nor does it include fragmental rock fall or rigid movements of relatively intact blocks of rock.

In consideration of the degree of complexity of the landslide dynamics, the cases are grouped as follows:

Group A - Calibration cases
1. Dam-break scenario
2. Laboratory test of dry sand flow prepared by Swiss Federal Institute of Technology, Lausanne
3. USGS flume test
Group B - Less sophisticated cases
4. Shum Wan Landslide, Hong Kong
5. Fei Tsui Road Landslide, Hong Kong
6. Sham Tseng San Tsuen Debris Flow, Hong Kong
7. 1990 Tsing Shan Debris Flow, Hong Kong

Group C - More sophisticated cases
8. Frank Slide, Canada
9. Thurwieser Rock Avalanche, Italy
10. 2000 Tsing Shan Debris Flow, Hong Kong
11. Tate's Cairn Landslide, Hong Kong
12. Lo Wai Debris Flood, Hong Kong

Data available for each of the cases includes the following:

(1) DEM-PATH: A digital elevation model of a rectangular area on the slope, encompassing the outline of the landslide rupture surface, path and deposition area (or a contour map showing the elevations of the pre-landslide topography).

(2) DEM-SOURCE: A DEM representing the vertical thickness map of the landslide source (or a contour map showing the elevations of the post-landslide topography; for the debris floods, the inflow hydrograph of the debris flow will be given, along with debris concentration and the specified location of the inflow). Source volume is understood here as the volume between the rupture surface and the original pre-slide ground surface. Both DEM’s will be represented by regular grids of elevations, referenced to the same point of origin and with equal grid spacing and extent. Thus, adding DEM-SOURCE to DEM-PATH would produce a digital elevation model of the pre-failure slope surface. Note, in case of landslides that begin by sliding, the source thickness DEM has not yet been increased by a factor allowing for volume increase due to fragmentation.

(3) A brief description of the landslide. The description will include surficial and bedrock geology, engineering-geological description of the materials forming the landslide source, runout path and deposit, areal distribution of different material covers along the runout path, comments on groundwater and surface water, land use and weather at the time of occurrence. Average volumetric bulking of the source material during detachment will be suggested. A number of photographs of each site will be provided. References for detailed site-specific landslide study reports, where available, are given.

(4) A map showing the outline of the final landslide deposits and the outline of any material entrainment areas along the runout path. Where possible, thickness distribution of final deposits and entrainment zones eroded by the landslide will be provided in the form of contours, or at least as spot values.

(5) A summary of all existing information regarding the behaviour of the landslide will be given, particularly observed or estimated movement velocities or movement duration, as far as available.

2.2 The Analysis

The participants are invited to assemble a dynamic model for each of the selected cases,
based on the above information. The appropriate rheological type (or model) and the corresponding material properties of the flowing mass are to be selected by each participating group based on an optimized back-analysis.

For the case at Tate’s Cairn (No. 11), in addition to undertaking a back-analysis of the 2005 debris flow, the participants are additionally invited to undertake a forward prediction of debris travel distance, debris velocity and runout paths for an impending failure involving the detachment of about 10000 m$^3$ of material from the distressed hillside given the presence of significant tension cracks on the hillside above the 2005 landslide source area. The results of the forward prediction are of relevance for the assessment of the necessary risk mitigation measures to protect the road and village houses below the distressed hillside. The forward prediction may be done in a deterministic or probabilistic manner. The case provides an opportunity for testing the capability of different modelling techniques and benchmarking the modelling results. Two years ago, this case posed a challenge to engineers in Hong Kong in an emergency situation. After the Tate's Cairn Landslide in 2005, detailed investigation revealed that there could be an impending failure involving a 10000 m$^3$ detachment from the distressed hillside. Debris mobility modelling was undertaken to establish the possible extent of potential consequence of this impending failure and provided useful information for the design of our emergency response actions.

2.3 Output

The participants should provide detailed plots of the flowing mass at intervals during motion and after stoppage. Further, they should provide summaries of movement velocities and direction vectors at various times. They should provide a brief discussion of each case specifying the rheological models and parameters used in the optimized back-analyses and any special provisions or assumptions used in the analyses.

The participants should further provide a brief description of the theory used in their models, with appropriate references. Also useful would be information on the computing resources used, e.g. computer, operating system, programming language and amounts of computing resources involved.

The output results should be summarized in succinct, illustrated reports on each case history. The reports may be in electronic (.pdf format) or hard copy.

The output data will be used by the selected participants to prepare their presentations at the Forum and by the Review Committee in preparing its summary report, which will also be presented at the Forum.
THE 2005 TATE’S CAIRN DEBRIS FLOW: BACK-ANALYSIS, FORWARD PREDICTIONS AND A SENSITIVITY ANALYSIS

José Cepeda
International Centre for Geohazards
and Department of Geosciences, University of Oslo

Abstract: In August 2005, a 1,000 m$^3$ debris flow in Tate’s Cairn (Hong Kong) stopped 200 m upstream of Kwun Ping Road and the Kwun Yam Shan and Kwun Yam Fa Yuen neighbourhoods. A back-analysis of this event and forward predictions of a potential debris flow initiating from a nearby 10,000 m$^3$ distressed zone were done with the computer programs DAN3D and FLO-2D. The back-analysis indicated that the DAN3D implementation of the Voellmy model matches the observations for an apparent friction angle of 15º and a turbulent coefficient of 1000 m/s$^2$. The quadratic model implemented in FLO-2D was not able to match both the observed trail and velocities simultaneously. The forward predictions indicate that Kwun Ping Road and some houses at Kwun Yam Shan are within the impact area of scenarios defined by the Review Committee for this Benchmarking Exercise to have frequencies of occurrence of 10% and 15%. The 10% scenarios also reach a few houses at Kwun Yam Fa Yuen. Scenarios with a 25% frequency of occurrence do not reach these locations. A sensitivity analysis of smoothed and non-smoothed DEMs showed that the simulated run-out is not significantly influenced, but differences in velocities and discharges can be as high as 40% and 100%, respectively.

INTRODUCTION
This paper presents the back-analysis of a debris flow that occurred in Tate’s Cairn, Hong Kong, in August 2005, and forward predictions of a potential 10,000 m$^3$ event at the same location. The 2005 event involved approximately 1,000 m$^3$ of sediments and stopped about 200 m from an inhabited area. This area located downstream from the maximum reach of the 2005 event may be at risk from a larger event. The simulations of the flow dynamics were performed using the computer programs DAN3D (McDougall 2006) and FLO-2D (FLO-2D Software Inc. 2006). This paper presents a brief summary of the models used for the simulations, a description of the 2005 event and the current potential threat, a detailed account of the methodology used for the simulations, the results and a sensitivity analysis using two different digital elevation models (DEMs).

MODELS
The programs used in the simulations were DAN3D, developed at the University of British Columbia (McDougall & Hungr 2004, 2005; McDougall 2006), and FLO-2D developed and commercialised by FLO-2D Software Inc. (O’Brien et al. 1993; FLO-2D Software Inc. 2006). Both programs are based on depth-averaged formulations of the equations of conservation of mass and momentum in a flow.

The DAN3D Program
DAN3D is based on a Lagrangian formulation that discretises the flow in a number of particles representing bed-normal columns of flow. The values of the field variables for each particle are calculated at each time step using an interpolation technique based on Smoothed
Particle Hydrodynamics (SPH) (Gingold & Monaghan 1977; Lucy 1977). The internal stresses are functions of the internal shear strains and are bounded by active and passive states. In the beta-version of the program used in the simulations, the calculation of the basal shear stresses has two choices of models: a Voellmy model (Eq. [1]) that consists of a frictional term and a velocity-dependent turbulent term; and a purely frictional model (Eq. [2]) that may account for dry conditions and for conditions with pore water pressure at the flow-bed interface. The equations for these models may be written as follows:

\[
\tau = \sigma \tan \phi_{\text{app}} + \frac{\gamma v^2}{\xi} \quad [1]
\]

\[
\tau = (1-r_u) \sigma \tan \phi \quad [2]
\]

where \( \tau \) is the bed shear stress; \( \sigma \) is the bed normal total stress; \( \phi_{\text{app}} \) is an apparent friction angle; \( \gamma \) is the unit weight of the flowing mass; \( v \) is the depth-averaged flow velocity; \( \xi \) is the turbulent coefficient; \( r_u \) is the pore-pressure ratio which is equal to the bed pore water pressure divided by the bed normal total stress; and \( \phi \) is the dynamic basal friction angle. Bed pore-water entrainment can be simulated after defining an entrainment zone, a maximum depth of supply material and an average growth or erosion rate, \( E_s \), defined as:

\[
E_s = \ln \left( \frac{V_f/\gamma_v}{V_0} \right) \quad [3]
\]

where \( V_0 \) and \( V_f \) are respectively the estimated total volume of the flowing mass before and after entrainment and \( S \) is the approximate average path length of the entrainment zone.

The FLO-2D Program

FLO-2D is an Eulerian formulation with a finite difference numerical scheme that requires the specification of an input hydrograph (i.e. a discharge time history) as a boundary condition at an uppermost location along the debris channel. The internal stresses are isotropic. The basal shear stresses are calculated using a quadratic model defined as follows:

\[
\tau = \tau_y + \frac{K \eta v}{8 h} + \frac{\gamma n_{\text{ad}}^2 v^2}{h^{1/3}} \quad [4]
\]

where \( \tau_y \) is the yield stress; \( K \) is a dimensionless resistance parameter that is set to 24 for laminar flow in smooth, wide, rectangular channels, but increases with roughness and irregular cross section geometry; \( \eta \) is the flow viscosity; \( h \) is the flow depth; \( n_{\text{ad}} \) is an empirically modified Manning’s \( n \) value; and the other variables are defined as previously. The yield stress and the viscosity are calculated respectively as \( \alpha_1 e^{\beta_1 C_V} \) and \( \alpha_2 e^{\beta_2 C_V} \), where \( C_V \) is the sediment concentration by volume; and \( \alpha_1, \beta_1, \alpha_2 \) and \( \beta_2 \) are regression constants obtained from the correlation of results of laboratory experiments.

THE 2005 TATE’S CAIRN DEBRIS FLOW: DESCRIPTION AND CONTEXT

The description presented in this section is mainly based on the detailed report prepared by Maunsell Geotechnical Services Ltd. (2007).
In August 2005, a landslide with an estimated volume of about 2,500 m$^3$ was triggered at Tate’s Cairn, Hong Kong. Approximately 1,000 m$^3$ detached from the release area and travelled along a drainage line turning into a mobile debris flow. The debris reached a travel angle of about 23º (i.e. a run-out ratio of 0.42), stopping approximately 200 m upstream of an inhabited area. Apart from the temporary disruption of a foot trail at the toe of the source area, neither damages nor casualties were reported for this event. Figures 1(a) and 1(b) show respectively a plan view of the site and a longitudinal section along the path of the flow.

Detailed field investigations following the August 2005 event revealed the presence of a distressed hillside with an estimated volume of 10,000 m$^3$ located directly behind the scarp of the 2005 release area and extending on the southwest direction. The potential detachment of this larger slide raised concern as to whether the inhabited areas farther downstream may be at risk. Particularly, the areas of interest are Kwun Ping Road, and the houses located in Kwun Yam Shan and Kwun Yam Fa Yuen (see Figure 1(a)).

The geological units in the site area are shown in Figure 1(a) and comprise hornfels ($m$), feldsparphyric rhyolite ($rf$), fine-grained granite ($gf$), and debris flow deposits ($Qd$) consisting of sand, gravel, cobbles and boulders in a clay/silt matrix. The debris flow deposits ($Qd$) are located downstream from the reach of the 2005 event.

Concerning past instabilities within the channel of the 2005 event, the Natural Terrain Landslide Inventory (NTLI) and Enhanced NTLI (ENTLI) landslide inventories in Hong Kong report a landslide that occurred before 1963, possibly in 1961 or 1962 (herein referred to as the pre-1963 event). This slide initiated directly below the toe of the 2005 release area and seemed to have travelled only along the ephemeral drainage line without entering the channel bend at the junction with the stream course (see Figure 1).

The surface of rupture of the 2005 slide appears to have been primarily located along an interface between colluvium deposits and an underlying saprolite. The deepest part of the rupture passed through a completely decomposed tuff (CDT). Consolidated-undrained (CU) triaxial tests were performed on the colluvium and the CDT, yielding $c' = 3$ kPa, $\phi' = 30^\circ$ and $c' = 9$ kPa, $\phi' = 37^\circ$ respectively for the peak states (Maunsell Geotechnical Services Ltd. 2007).

For the 2005 debris flow, the mass balance schedule indicates a very low debris entrainment ratio (less than 5%), part of which was a 20 m$^3$ slide released independently (i.e. apparently not due to undercutting of the channel banks by the debris flow) between chainages 173 and 188 (see Figure 1(b)).

**METHODOLOGY**

The back-analysis of the 2005 event was performed using DAN3D and FLO-2D. The forward predictions for the potential 10,000 m$^3$ slide were performed using only DAN3D because the Voellmy model, which was adopted for this part, cannot be implemented in FLO-2D.

DAN3D and FLO-2D require different sets of parameters that control the numerical performance of the models in terms of convergence, accuracy and efficiency. The numerical parameters for DAN3D are the number of particles, the time step, the particle smoothing coefficient, the velocity smoothing coefficient, and the stiffness coefficient. In the case of FLO-2D, the parameter that mainly controls the numerical performance is the grid spacing. In
the simulations using FLO-2D, the grid spacing was taken as 5 m, which corresponds to the grid size in the digital elevation models provided by the Review Committee of the Benchmarking Exercise.

**Back-analysis: General**

The following field observations were used for the calibration of the simulations:

- The outline or trim line of the debris flow as indicated in Figure 1(a).
- The measured super-elevation at the channel bend where the ephemeral drainage line intersects the stream course (Figure 1(a)). The velocity estimated from the super-elevation equation (see, e.g. Chow 1959, and Johnson 1970) is 7.6 m/s and corresponds to a super-elevation of 4 m (for elevations of 6 and 2 m on the sides of the channel), a downslope angle of 23.6º, a width of 21 m, and a radius of curvature of 33.5 m.
- The distribution of final deposits along the debris flow path as presented in Figure 1(b).
- No entrainment was considered since the very low debris entrainment ratio (less than 5%) is considered too small to produce significant differences in the results of the analyses.

**Back-analysis using DAN3D**

Regarding the model for the basal shear stresses, the friction model has proven to work well with very small natural debris flows (volumes of less than 1,000 m³), while the Voellmy model has proved suitable for larger volumes (Ayotte & Hungr 2000). In consistent with the guidelines for the forward predictions from the Review Committee for the Benchmarking Exercise (Table 1), the Voellmy model was used in the simulations of the 2005 event. The sets of parameters are presented in Table 2. An internal friction angle of 35º was employed, which was also used in all the debris flow cases back-analysed by McDougall (2006).

In the numerical simulations with DAN3D, the number of particles, \( N \), should be large enough to ensure the accurate simulation with respect to flow spreading, junctioning and branching. One guideline for the selection of this parameter is the experience from other simulations. McDougall (2006) simulated 8 debris flows and avalanches ranging in volume from 4,200 to 735,000 m³ with 2,000 particles and from 1.4 to 130 million m³ with 4,000 particles. Given this data and the volumes to be simulated in the back-analysis and the forward predictions, the number of particles was set to 2,000 for all the simulations.

The time step should be sufficiently small as not to influence the results of the simulation. A series of simulations was performed with varying time steps, starting with a value of 2 s, and then reducing by half for smaller time steps. The simulations were done with a flow volume of 10,000 m³, an apparent friction angle of 8º and a turbulent coefficient of 500 m/s² since this combination produces the longest runout in the set of cases with no entrainment (see Table 1). Three output values were calculated as indicators of the results: the area of impact (i.e. the area of the simulated debris trail), the average of the maximum flow depth and the average of the maximum velocity. These values were calculated for the full length of the simulated debris path and for the segment of the debris path crossing the area of interest (the Kwun Ping Road and the houses at Kwun Yam Shan and Kwun Yam Fa Yuen). The results are presented in Figure 2, and indicate that a time step less than or equal to 0.05 s produces a stable solution, hence this was selected as the time step for all simulations.

The velocity smoothing coefficient, \( C \), increases numerical stability and improves the behaviour of the model in channelised reaches. However, it introduces some numerical damping and hence it is not appropriate to use high values. McDougall (2006) suggests a
value up to about 0.01 to be appropriate. In order to select the value of \( C \) for the experiments, simulations were run for the back-analysis scenario and the set of parameters for case 2(b) in Table 1. Two different choices of \( C \) were used: 0 and 0.01. Figures 3 and 4 show the resulting time histories of maximum velocities and front position. The results show that in both scenarios a numerical noise of high frequency occurs in the maximum velocity starting from around \( t = 100 \) s (see Figure 3). This high frequency noise is more pronounced in the \( C = 0 \) case than in the \( C = 0.01 \). A practical implication of this high-frequency noise is shown in Figure 4, where the runout distance is almost doubled in the \( C = 0 \) case after the high frequency noise in the maximum velocity is initiated at about 100 s. It can also be seen that the runout is only slightly increased in the \( C = 0.01 \) case, especially after \( t = 700 \) s. Based on the above, the value of \( C = 0.01 \) was selected for the simulations in order to minimise the consequences of this numerical noise.

The values of the remaining parameters in the DAN3D model, the particle smoothing coefficient and the stiffness coefficient were set to be equal to 4 and 200, respectively, as suggested by McDougall & Hungr (2004, 2005) and McDougall (2006).

In order to compare the observed velocity at the channel bend with the simulated velocities, it was necessary to calculate the average flow velocity that corresponds to the maximum super-elevation. This velocity is not included in the output results of DAN3D, so a post-processing step involved the calculation of the time history of the following variables across a user-specified cross-section (see Figure 5 for illustration of variables): farthest point across flow section \( (x_{max}) \), elevation at \( x_{max} \) \( (z_{x_{max}}) \), average flow velocity, discharge, and average elevation of flow surface. It is worth noting that the comparison of the maximum velocity grids generated by DAN3D and the field velocity back-calculated at a channel bend involves the potential error of choosing a maximum velocity (Figure 5, left) that may not necessarily correspond to the conditions at the time of the maximum super-elevation (Figure 5, right).

**Back-analysis using FLO-2D**

The back-analysis using this program involved the calculation of an input hydrograph followed by simulations using a number of sets of model parameters. The input hydrograph was calculated as follows:

- The material in the source area was modelled as a frictional material using DAN3D.
- A debris flow hydrograph was calculated at the toe of the release area.
- Two types of water hydrographs were calculated using variable and constant sediment concentrations.
- Each hydrograph was distributed in 5 cells (see Figure 6) located along the cross-section in proportion to the maximum flow depths observed in the DAN3D output.

The model parameters for the source area were: angle of friction of 30° and pore-water pressure ratio of 0.5 (i.e. assuming groundwater level at the terrain surface). The value of the friction angle is assumed for a residual state in the colluvium. Based on shear box tests on colluvial soils performed by Lacerda & Silveira (1992) and reported by Lacerda (2007), the shear stresses for the residual state are approximately equal to the peak state, and hence the friction angle obtained by Maunsell Geotechnical Services Ltd. (2007) was adopted for this simulation. The debris flow hydrograph was calculated along a cross-section passing through the coordinates (840027, 824520) and (840098, 824484).
Two different scenarios of sediment concentrations were considered:

- **Variable sediment concentration.** A time history of sediment concentration was produced based on the shape of the hydrograph. This is in consistent with observations that the peaks in debris flow hydrographs correspond to high sediment concentrations, while the raising and final tails have a more diluted composition (Bertolo & Wieczorek 2005; FLO-2D Software Inc. 2006). This scenario tries to reproduce the distribution of sediment concentration when the event occurs during heavy precipitation which may produce a dilution in the raising and falling tails of the hydrograph. Two days before the 2005 Tate’s Cairn debris flow was first observed, a peak hourly rainfall of 40 mm was recorded in a station about 1.7 km southwest of the landslide site (Maunsell Geotechnical Services Ltd. 2007). For the design of the water hydrograph, the maximum and minimum concentrations were set to 0.53 and 0.20, respectively, which bracket the sediment concentrations for mudflows and mud floods (FLO-2D Software Inc. 2006).

- **Constant sediment concentration.** This scenario is consistent with a debris flow initiated during a period with little or no precipitation. Within two days prior to the first observation of the 2005 debris flow, there were continuous periods with no precipitation lasting between 2 and 5 hours. Sediment concentrations by volume ranging from 0.3 to 0.48 were used for this scenario.

The model parameters for the calculation of the yield strength and the viscosity were selected from the set of materials presented by O’Brien and Julien (1988) and suggested in the user manual of FLO-2D. Figures 7(a) and 7(b) show the variation of yield stress and viscosity as a function of sediment concentration for the complete set of materials. Figure 7(c) presents the flow curves for all the materials for a sediment concentration of 0.4.

Finally, the Manning’s $n$-values that characterise the roughness of the terrain need to be defined. Four values were selected: 0.04, 0.07, 0.1, and 0.2. These values correspond to the lower bound for open ground with no debris (0.04), the limit between open ground with debris and without debris (0.1), and the upper bound for open ground with debris (0.2). The value of 0.07 is taken as an intermediate case between $n = 0.04$ and $n = 0.1$. The above definitions are established in the FLO-2D user’s manual and are based on guides and manuals published by the United States Army Corps of Engineers.

The sets of parameter combinations that were applied in the FLO-2D simulations are summarised in the first four columns in Table 5. In this table, the light- and dark-grey cells correspond respectively to the Aspen Pit 1 and Glenwood 2 rheologies (see Figure 7) which bound the flow curves presented in Figure 7(c).

**Forward Predictions for the Larger Slide**

As mentioned earlier, only DAN3D was used in this part of the exercise with the same numerical parameters as in the back-analysis. The model parameters that were adopted for the forward predictions are listed in Table 1. These parameters and their associated frequencies of occurrence (last column of Table 1) were defined by the Review Committee for the Benchmarking Exercise, but the corresponding exposure time was not defined.

In the cases with a 100% debris entrainment ratio, four different scenarios were considered:

- **Scenario I:** entrainment only upstream from the channel bend. It is assumed that only the debris of the pre-1963 event is entrained. This implies that prior to the occurrence of the large slide, all the 2005 debris has already been removed and transported downstream due
to erosion by runoff water.

- Scenario II: entrainment only along the 2005 debris trail. Here, the entrained material is supplied mainly by the debris of the pre-1963 and the 2005 events.
- Scenario III: entrainment from the toe of the distressed area down to the extent of the quaternary deposits. It is assumed that the supply of erodible bed includes debris from the pre-1963 and 2005 events and the debris flow deposits located downstream from the reach of the 2005 landslide (see Figure 1(a)).
- Scenario IV: entrainment only within extent of quaternary deposits. Here, the debris from the pre-1963 and 2005 events has been transported downstream, and hence the supply of bed material is located downstream from the reach of the 2005 debris flow.

The two input parameters that need to be defined after specifying the extent of the erodible zone are the depth of the erodible material and the erosion or growth rate. A constant depth of 5 m was assigned to the erodible zone. This value is taken from the maximum thickness of material in the potential release area within the distressed hillside. The erosion or growth rate (Eq. [3]), defined according to McDougall & Hungr (2005), is listed for each entrainment scenario in Table 3.

RESULTS

Back-analysis of the 2005 Debris Flow
The simulations of the 2005 debris flow using DAN3D are presented in Figures 8 to 11. Table 2 summarises the simulated flow velocities at the channel bend at the time of the maximum superelevation. From these results, the combination of parameters that best corresponds to the field observations is parameter set D, which is characterised by an apparent friction angle of 15º and a turbulent coefficient of 1000 m/s².

Table 4 and Figure 12 present the results used for the design of the inflow hydrograph for the FLO-2D simulations. Table 5 summarises the results of these simulations. The results using parameter set 11 are the closest to the observations in terms of runout distance, even though the maximum simulated velocity in the channel bend (1 m/s) is well below the observed value of 7.6 m/s. The yield strength and viscosity for this set are 240 Pa, and 13 Pa•s, respectively. Parameter set 16 is the combination that best predicts the velocity in the channel bend, but it overestimates the runout distance by more than 50 m. The yield strength and viscosity for set 16 are respectively 6.6 Pa and 0.8 Pa•s.

Forward Predictions for the Large Slide
Figure 13 presents the results for the forward prediction of the 10,000 m³ debris flow considering no entrainment along the path. Cases 1(b) and 2(b) (with frequencies of occurrence of 10% and 15%, respectively) impact the area of interest, mainly Kwun Ping Road. The maximum runout for Case 1(b) extends beyond the limit of the terrain model. The same occurred also for the erosion cases (see Type-1 cases in Figure 14). Case 3(b) produces a runout distance comparable to the 2005 debris flow. This latter case was not further considered for the predictions that included erosion since it was not regarded relevant for the inhabited areas and the road.

Figure 14 presents the results of the predictions that consider debris entrainment along the path. Scenario I is always the most threatening in terms of velocity and flow depth. Scenarios II, III and IV show gradually lower velocities and flow depths. Scenario IV gives results very similar to those for the no-erosion scenarios.
SENSITIVITY ANALYSIS

This section presents a sensitivity analysis of the simulations considering differences in the terrain model while keeping all the other parameters constant. The analysis was performed by comparing the results using two different terrain models or digital elevation models (DEM). On 7 May 2007, the Review Committee of the Benchmarking Exercise provided a terrain model, herein referred to as the “old” DEM. Later, on 2 July 2007, the Committee provided a “new” DEM – the one used in all the previous simulations in this paper – with an extended coverage downstream to the north of the area of interest. Figures 15(b) and 15(c) show that the “new” model is a smoothed version of the “old” model. The two DEMs present differences in elevation ranging from -7.8 to 5.7 m (Figure 15(a)).

The streamlines calculated from the two terrain models show differences in the vicinities of the channel bend (Figures 15(b) and 15(c)). The “old” model is the one that best matches the channel bend. The differences in the DEMs have a direct consequence on the simulated debris path (dashed contours in Figures 15(b) and 15(c)). The most significant differences in the simulations using the two terrain models are in the values of discharges and velocities in the channel bend (Table 2). For parameter set D, the simulated velocity with the “old” DEM is 30% higher than with the “new” DEM, and the simulated discharge with the “old” DEM is almost double of that with the “new” DEM. Finally, the pattern of final deposition also shows significant differences between the two terrain models (Figure 16).

CONCLUSIONS

- The back-analysis of the August 2005 debris flow indicates that in the DAN3D implementation of the Voellmy model, the parameters that produce the closest agreement with the observed debris trail and velocities are an apparent friction angle of 15° and a turbulent coefficient of 1,000 m/s².
- The quadratic model implemented in FLO-2D is not able to produce results that match both the observed trail and velocities simultaneously. For this model, the set of parameters that best simulates the debris trail significantly underestimates the velocity, and those that best simulate the flow velocities, significantly overestimate the runout.
- The forward predictions for the 10,000 m³ debris flow indicates that Kwun Ping Road, and some houses at Kwun Yam Shan are within the impact area of the scenarios with frequencies of occurrence of 10% and 15%.
- The scenarios with a 10% frequency of occurrence also impact a few houses located in Kwun Yam Fa Yuen.
- The scenario with frequency of occurrence of 25% does not impact any of these locations.
- Kwun Ping Road may be affected with maximum flow depths in the range of 2 to 3 m, and maximum velocities ranging between 5 and 10 m/s for the 10% frequency of occurrence scenario. The corresponding ranges for the 15% scenario are 2 – 3 m, and 2 – 5 m/s.
- The houses at Kwun Yam Shan which are closest to the stream course may be affected by maximum flow depths ranging from 2 to 3 m and maximum velocities from 5 to 10 m/s for the 10% frequency of occurrence scenario. The corresponding ranges for the 15% scenarios are 1 – 2 m, and 2 – 5 m/s.
- In the 10% scenario, the northernmost houses in Kwun Yam Fa Yuen located closest to the stream course may be affected by maximum flow depths of up to 1 m, and maximum velocities ranging between 5 and 10 m/s.
- A sensitivity analysis using smoothed and non-smoothed DEMs shows that the simulated run-out length and debris trail are not significantly influenced, but the relative differences in velocities and discharges at the channel bend at the time of the maximum
superelevation can be as high as 40% and 100%, respectively.

REFERENCES

ACKNOWLEDGEMENTS
The Organizing and Technical Committees of the 2007 Hong Kong Landslide Forum are gratefully acknowledged for the opportunity to participate in this exercise. The "International Centre for Geohazards" (ICG) and the Norwegian Quota Scheme provided financial support. Prof. Oldrich Hungr at the University of British Columbia kindly provided ICG/NGI (Norwegian Geotechnical Institute) with a version of DAN3D for non-commercial applications. Byron Quan collaborated with the preparation of GIS features. Comments from Kaare Høeg, Farrokh Nadim, Bjørn Kalsnes and Peter Gauer have contributed to improve the contents and presentation of this paper. This is ICG article No. 193.
Table 1: Model parameters adopted for the forward predictions (following guidelines from the Review Committee for the Benchmarking Exercise)

<table>
<thead>
<tr>
<th>Case</th>
<th>Debris entrainment ratio</th>
<th>Voellmy model parameters</th>
<th>Frequency of occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Apparent friction angle</td>
<td>Turbulent coefficient (m/s²)</td>
</tr>
<tr>
<td>1(a)</td>
<td>100%</td>
<td>8º</td>
<td>500</td>
</tr>
<tr>
<td>1(b)</td>
<td>0%</td>
<td>8º</td>
<td>500</td>
</tr>
<tr>
<td>2(a)</td>
<td>100%</td>
<td>15º</td>
<td>1000</td>
</tr>
<tr>
<td>2(b)</td>
<td>0%</td>
<td>15º</td>
<td>1000</td>
</tr>
<tr>
<td>3(a)</td>
<td>100%</td>
<td>25º</td>
<td>1000</td>
</tr>
<tr>
<td>3(b)</td>
<td>0%</td>
<td>25º</td>
<td>1000</td>
</tr>
</tbody>
</table>

Table 2: Model parameters used for the back-analysis and simulated velocities and discharges at channel bend

<table>
<thead>
<tr>
<th>Parameter set</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apparent friction angle (º)</td>
<td>15</td>
<td>20</td>
<td>25</td>
<td>15</td>
<td>20</td>
<td>25</td>
</tr>
<tr>
<td>Turbulent coefficient (m/s²)</td>
<td>500</td>
<td>500</td>
<td>500</td>
<td>1000</td>
<td>1000</td>
<td>1000</td>
</tr>
<tr>
<td>Velocity(m/s)</td>
<td>5.8</td>
<td>3.6</td>
<td>1.5</td>
<td>7.3</td>
<td>4.6</td>
<td>2.5</td>
</tr>
<tr>
<td>Discharge (m³/s)</td>
<td>47</td>
<td>29</td>
<td>9.5</td>
<td>61</td>
<td>35</td>
<td>12</td>
</tr>
<tr>
<td>Velocity (m/s) [old DEM]</td>
<td>7.2</td>
<td>5.0</td>
<td>0.8</td>
<td>9.5</td>
<td>6.6</td>
<td>3.0</td>
</tr>
<tr>
<td>Discharge (m³/s) [old DEM]</td>
<td>96</td>
<td>53</td>
<td>8.9</td>
<td>120</td>
<td>73</td>
<td>8.4</td>
</tr>
</tbody>
</table>

Table 3: Erosion or growth rates, $\bar{E}_s$, for scenarios with 100% debris entrainment ratio

<table>
<thead>
<tr>
<th>Erosion scenarios</th>
<th>Average path length of entrainment zone (m)</th>
<th>Erosion rate (m⁻¹)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>166</td>
<td>0.0042</td>
</tr>
<tr>
<td>II</td>
<td>263</td>
<td>0.0026</td>
</tr>
<tr>
<td>III</td>
<td>946</td>
<td>0.00073</td>
</tr>
<tr>
<td>IV</td>
<td>683</td>
<td>0.0010</td>
</tr>
</tbody>
</table>

Table 4: Coordinates of inflow nodes for FLO-2D and distribution of total hydrograph

<table>
<thead>
<tr>
<th>Point number</th>
<th>Longitude (m)</th>
<th>Latitude (m)</th>
<th>Max. Depth (m)</th>
<th>Fraction of total hydrograph</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>840065</td>
<td>824500</td>
<td>0.15</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>840070</td>
<td>824495</td>
<td>0.54</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>840075</td>
<td>824495</td>
<td>0.53</td>
<td>0.3</td>
</tr>
<tr>
<td>4</td>
<td>840080</td>
<td>824490</td>
<td>0.45</td>
<td>0.25</td>
</tr>
<tr>
<td>5</td>
<td>840085</td>
<td>824490</td>
<td>0.13</td>
<td>0.07</td>
</tr>
</tbody>
</table>
Table 5: Summary of results of simulations using FLO-2D. The relative front position in the fifth column indicates the distance between the final position of the front and the location of the observed maximum runout (negative and positive values indicate under-prediction and over-prediction of runout, respectively).

<table>
<thead>
<tr>
<th>Set</th>
<th>Sediment concentration</th>
<th>Manning’s n-value</th>
<th>Rheology</th>
<th>Relative front position (m)</th>
<th>$v_{max}$ (m/s)</th>
<th>$v_{max}$ in bend (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Variable</td>
<td>0.1</td>
<td>Aspen Pit 1</td>
<td>+ 49</td>
<td>4.6</td>
<td>1.3</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td>Aspen mine source area</td>
<td>&gt; + 50</td>
<td>5.9</td>
<td>2.7</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td>Glenwood 2</td>
<td>&gt; + 50</td>
<td>7.2</td>
<td>4.2</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td>Glenwood 4</td>
<td>&gt; + 50</td>
<td>5.1</td>
<td>1.6</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>Aspen Pit 1</td>
<td>+ 49</td>
<td>3.9</td>
<td>1.2</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>0.2</td>
<td>Aspen mine source area</td>
<td>&gt; + 50</td>
<td>4.4</td>
<td>2.0</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
<td>Glenwood 2</td>
<td>&gt; + 50</td>
<td>4.6</td>
<td>2.5</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td>Glenwood 4</td>
<td>&gt; + 50</td>
<td>4.0</td>
<td>1.4</td>
</tr>
<tr>
<td>9</td>
<td>Constant = 0.30</td>
<td>0.04</td>
<td>Aspen Pit 1</td>
<td>&gt; +50</td>
<td>9.3</td>
<td>3.2</td>
</tr>
<tr>
<td>10</td>
<td>Constant = 0.35</td>
<td>0.04</td>
<td>Aspen Pit 1</td>
<td>&gt; +50</td>
<td>6.6</td>
<td>1.4</td>
</tr>
<tr>
<td>11</td>
<td>Constant = 0.37</td>
<td>0.04</td>
<td>Aspen Pit 1</td>
<td>+9</td>
<td>5.6</td>
<td>1.0</td>
</tr>
<tr>
<td>12</td>
<td></td>
<td>0.04</td>
<td>Aspen Pit 1</td>
<td>- 32</td>
<td>4.5</td>
<td>0.6</td>
</tr>
<tr>
<td>13</td>
<td></td>
<td></td>
<td>Aspen Pit 2</td>
<td>&gt; +50</td>
<td>10.4</td>
<td>4.4</td>
</tr>
<tr>
<td>14</td>
<td></td>
<td></td>
<td>Aspen natural soil</td>
<td>+34</td>
<td>5.8</td>
<td>1.1</td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
<td>Aspen mine fill source</td>
<td>+34</td>
<td>5.8</td>
<td>1.1</td>
</tr>
<tr>
<td>16</td>
<td></td>
<td>0.04</td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>11.1</td>
<td>7.3</td>
</tr>
<tr>
<td>17</td>
<td></td>
<td></td>
<td>Glenwood 3</td>
<td>&gt; +50</td>
<td>10.4</td>
<td>4.4</td>
</tr>
<tr>
<td>18</td>
<td></td>
<td></td>
<td>Glenwood 4</td>
<td>- 37</td>
<td>4.1</td>
<td>0.5</td>
</tr>
<tr>
<td>19</td>
<td>Constant = 0.40</td>
<td>0.07</td>
<td>Aspen Pit 2</td>
<td>&gt; +50</td>
<td>8.2</td>
<td>3.7</td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>8.6</td>
<td>5.3</td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>0.1</td>
<td>Aspen Pit 1</td>
<td>- 32</td>
<td>4.2</td>
<td>0.5</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td></td>
<td>Aspen Pit 2</td>
<td>&gt; +50</td>
<td>6.9</td>
<td>3.2</td>
</tr>
<tr>
<td>23</td>
<td></td>
<td></td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>7.2</td>
<td>4.2</td>
</tr>
<tr>
<td>24</td>
<td></td>
<td>0.2</td>
<td>Aspen Pit 1</td>
<td>- 32</td>
<td>3.6</td>
<td>0.5</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
<td>Aspen Pit 2</td>
<td>&gt; +50</td>
<td>4.5</td>
<td>2.2</td>
</tr>
<tr>
<td>26</td>
<td></td>
<td></td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>4.6</td>
<td>2.4</td>
</tr>
<tr>
<td>27</td>
<td>Constant = 0.45</td>
<td>0.04</td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>11.0</td>
<td>6.2</td>
</tr>
<tr>
<td>28</td>
<td>Constant = 0.48</td>
<td>0.04</td>
<td>Glenwood 2</td>
<td>&gt; +50</td>
<td>10.9</td>
<td>5.7</td>
</tr>
</tbody>
</table>
Figure 1: General description of the Tate’s Caim case. (a) Site layout plan including geological units (see description in text); grid spacing is 100 m. (b) Longitudinal section along debris trail and mass balance schedule (Maunsell Geotechnical Services Ltd. 2007).
Figure 2: Results of simulations as a function of time step along full path and in interest area (see definition in text). (a) Area of impact along full path (left axis and circle markers) and only in interest area (right axis and square markers). (b) Average of maximum flow depth (left axis and circle markers) and average of maximum velocity (right axis and square markers); solid and dashed lines are for full path, and interest area, respectively.

Figure 3: Simulated maximum velocities against time for two different values of the velocity smoothing coefficient, $C$. (a) Using $C = 0$. (b) Using $C = 0.01$.

Figure 4: Simulated front position (latitude) against time for two different values of the velocity smoothing coefficient, $C$
Figure 5: Cross section of flow at channel bend. Left: time of maximum velocity at shaded location. Right: time of maximum super-elevation at channel bend.

Figure 6: Cells (points) 1 to 5 used as inflow nodes for the FLO-2D analysis. Grayscale ramp grid is thickness of material in release area. Black cells are location of ridge calculated from DEM. Thick outline is the trail of the 2005 debris flow.
Figure 7: Model parameters for FLO-2D. (a) and (b) Yield stress and viscosity as a function of sediment concentration by volume, $C_v$. (c) Flow curves for a sediment concentration of 0.4 based on data by O’Brien & Julien (1988).
Figure 8: Simulated maximum depths (top) and velocities (bottom) in back-analysis. Sets of model parameters A to F are ordered from left to right. See Table 2 for definition of each set. Thick outline is the observed August 2005 debris trail.
Figure 9: Cumulative volume of final deposits in simulations (lines) and from field observations (line with circle markers). The plot of field observations corresponds to the mass balance schedule presented in Figure 1(b).

Figure 10: Time history of flow through cross-section in channel bend (intersection of ephemeral drainage line and stream course in Figure 1(a) using set D of parameters (Table 2). \( xp_{\text{max}} \) and \( zp_{\text{max}} \) are defined in Figure 5, and \( z_{\text{avg}} \) is the mean elevation of the flow surface.
Figure 11: Evolution of simulation over time for set D of parameters using DAN3D. The depth plots also show the elevation contours. The horizontal velocity plots show the vectors for only 50 particles, not for all the 2000 particles used in the simulation.
Figure 12: Hydrograph of debris flow and water (left axis), and sediment concentration by volume, $C_V$ (right axis).

<table>
<thead>
<tr>
<th>Case 1(b): 10% frequency of occurrence</th>
<th>Case 2(b): 15% frequency of occurrence</th>
<th>Case 3(b): 25% frequency of occurrence</th>
<th>Case 1(b): 10% frequency of occurrence</th>
<th>Case 2(b): 15% frequency of occurrence</th>
<th>Case 3(b): 25% frequency of occurrence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum depths (m)</td>
<td>Maximum velocities (m/s)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High: 7.8</td>
<td>High: 18.6</td>
<td></td>
<td>Low: 0</td>
<td>Low: 0</td>
<td></td>
</tr>
</tbody>
</table>

Figure 13: Maximum depths and velocities in forward prediction for no-entrainment cases. See description of cases in Table 1.
Figure 14: Results of forward predictions. Types 1 and 2 cases have frequencies of occurrence of 10% and 15%, respectively (see Table 1).
Figure 15: Comparison of “Old” and “New” digital elevation models. (a) Differences in elevation after subtracting “New” DEM from “Old” DEM. (b) and (c) Streams and simulated debris trails (using set D in Table 2) obtained from “Old” DEM (B) and “New” DEM (C).

Figure 16: Cumulative volume of final deposits against latitude. Solid lines: “New” terrain model. Dashed lines: “Old” terrain model. Circle markers with line: final deposits from field observations.
ANALYSIS OF HONG KONG DEBRIS FLOW WITH AN ENERGY BASED MODEL

Dave Chan, Norbert Morgenstern, Dat Tran and Xiaobo Wang
Department of Civil and Environmental Engineering
The University of Alberta, Canada

Abstract: This paper describes the application of a new formulation in debris flow modelling for analyzing rapid mass movement in Hong Kong. The formulation is based on energy balance considering conversion of potential energy to kinetic energy and dissipation of internal energy. The energy equation is used in formulating the equation of motion because it is easier to account for various sources of energy dissipation. Internal energy is taken into account by calculating the deformation of the flowing mass using a sliced based dynamic model. The model has been used in studying 10 case histories in a debris flow benchmarking exercise. These cases include calibration cases with known analytical solutions, simple flow cases and complex flow situations. The model is robust and it is able to model the debris flow of many of these cases.

INTRODUCTION
This paper introduces a new method in analyzing 10 cases of modeling of debris flow experiments and case studies. It is part of a benchmarking exercise in the landslide forum held in Hong Kong between December 10 and 12, 2007. The cases are grouped into three categories; Group A – calibration cases, Group B – less sophisticated cases and Group C - more sophisticated cases. The analysis was carried out using a depth averaging slice based computer model with modifications to calculate internal energy dissipation and internal interslice forces. Energy balance is used in calculating the velocity of flow. Potential energy is converted to kinetic energy and internal energy dissipation of the flowing debris. The model satisfies the momentum equation and the energy conservation equation using the Mohr Coulomb friction model.

Three cases were analyzed in Group A: the analytical solution of Dam Break provide by Mangeney, one experimental study by EPFL and one small scale experiment of sand flow by USGS. Comparison of the results between the calculated and observed soil movement shows that the present model is able to analyze soil movement with reasonable accuracy. In most cases, the measured soil parameters were used except in one case where the soil parameters were changed slightly to obtain the best results.

Four cases were analyzed in Group B, Shum Wan Landslide, Fei Tsui Road Landslide, Sham Tseng San Tsuen Debris Flow and the 1990 Tsing Shan Debris Flow. The calculated and observed final debris profiles are compared in these cases. It is concluded that the present model is able to model movements reasonably because the final calculated soil profiles compare well with field observations.

Three cases were analyzed in Group C - the 2000 Tsing Shan Debris Flow, Tate's Cairn Landslide and Lo Wai Debris Flood. These cases are more complex and the movements of
soil were faster in comparison with Group B cases. As well, the debris bifurcates and flows in different channels for the case of the Tsing Shan 2000 debris flow.

The University of Alberta is one of the participants in the Benchmarking Exercise on Debris Runout and Mobility of the 2007 International Forum on Landslide Disaster Management, Hong Kong. In this exercise, we were provided with information on 12 cases. These cases are classified into three groups:

- **Group A - Calibration cases**
  1. Dam-break scenario
  2. Laboratory test of dry sand flow prepared by Swiss Federal Institute of Technology, Lausanne
  3. USGS Flume tests

- **Group B - Less sophisticated cases**
  4. Shum Wan Landslide, Hong Kong
  5. Fei Tsui Road Landslide, Hong Kong
  6. Sham Tseng San Tsuen Debris Flow, Hong Kong
  7. 1990 Tsing Shan Debris Flow, Hong Kong

- **Group C - More sophisticated cases**
  8. Frank Slide, Canada (not analyzed in the present study)
  9. Thurwieser Rock Avalanche, Italy (not analyzed in the present study)
  10. 2000 Tsing Shan Debris Flow, Hong Kong
  11. Tate's Cairn Landslide, Hong Kong
  12. Lo Wai Debris Flood, Hong Kong

**DESCRIPTION OF THE MODEL**
The model used in this paper is a slice based continuum model. The model has been developed at the University of Alberta by Wang (2008). It is a continuum model which satisfies the equation of motion, the continuity equation, the energy equation and constitutive description of the material. The flowing debris is divided into two dimensional slices and the governing equations are solved using an explicit time marching scheme. The Mohr Coulomb friction model is implemented in the computer program with separate angle of internal friction and basal friction.

The approach is based on a Lagrangian, moving mesh, finite difference scheme in which the flowing material is divided into quadrilateral cells in two dimensions. Boundary locations are determined for each time step. The depth of a cell is calculated from cell volume and boundary locations. Numerical simulations of flume experiments on dry granular flows in the past showed very good agreement between theoretical predictions and observation data (Savage & Hutter 1989, 1991; Wieland et al. 1999).

The formulation of the governing equation considers energy balance over a time interval $\Delta t$ which can be written as:

$$\frac{E_i^{t+\Delta t} - E_i^t}{\Delta t} = \dot{W}_i^t$$  \[1\]
\[ E_i = \frac{1}{2} m_i \mu_i^2 \]  
\[ \dot{W}_i = m_i g u_i \sin \theta_i - \frac{1}{2} m_i g h_i (e_{zz})_i + (P_{LU} \cos \theta_i)_i - (P_{RU} \cos \theta_i)_i - T_i \mu_i - \int \tau_{ij} e_{ij} \, dV \]  

where \( E_i \) is the kinetic energy of slice \( i \), \( \dot{W}_i \) is the sum of the rate of the work done by body force, surface traction, and energy dissipation due to deformation of slice \( i \). \( E_i \) and \( \dot{W}_i \) are determined from Eqs. [2] and [3], respectively.

The constitutive law and assumptions regarding interslice forces and deformation work are required for calculating the rate of work in Eq. [3]. The basal shear resistance can be determined based on the constitutive laws for materials. For Mohr-Coulomb materials, the basal resistance along the base of the slice can be expressed as:

\[ \tau = c + \sigma \tan \phi_b \]  

where \( \tau \) is shear stress, \( c \) is cohesion, \( \sigma \) is normal stress incorporating the effect of curvature of sliding path, and \( \phi_b \) is basal friction angle.

At time \( t = 0 \), the velocities and kinetic energy of slices are equal to zero. The acceleration of slice \( i \) can be determined from momentum conservation:

\[ m_i \frac{du_i}{dt} = m_i g \sin \theta_i - T_i + (P_{LU} - P_{RU}) \cos \theta_i \]  

The velocities and displacements of slices during the first time step can be calculated as long as accelerations are obtained from Eq. [5]. The motion of each slice can then be determined using equations of energy conservation with the Lagrangian difference scheme presented above. The computation proceeds until the maximum slice velocity is under the velocity threshold specified. Details of the analysis are developed in Wang (2008). It should be noted that this formulation assumes Rankine failure states for internal stresses as opposed to the commonly adopted Savage & Hutter relationships (1989).

**RESULTS OF THE ANALYSIS**

The parameters used for analyzing the debris flow cases are summarized in Table 1. Only 10 cases have been analyzed in this study, since 2 of the 12 cases involved rock slides. The present model is developed for soil movement, therefore only the soil cases were analyzed. Different material properties were used and the one that best fits the observed results using the current model are shown. For each case, the model calculates the movement of soils from its initial position to the final resting position. The program calculates the profile of the debris at different times. Soil profiles at selected times are plotted to see the flowing process. The velocity at different locations for the front of the debris is also plotted. In most cases, the velocities are zero when the debris has been arrested. However, for a couple of cases, the velocities are not zero, but they are small. The main part of the debris has come to rest and only the front is moving slowly. In the debris flow cases, only a selected number of cases are presented. Results of all the cases can be found in Chan et al. (2007).
Table 1: Summary of material parameters used in computer simulation

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Wt (kN/m³)</th>
<th>Best Fit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Material</td>
<td></td>
<td>$\phi$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Basal</td>
</tr>
<tr>
<td>Dam-break scenario</td>
<td>20</td>
<td>0</td>
</tr>
<tr>
<td>Dry sand flow experiment</td>
<td>15</td>
<td>35</td>
</tr>
<tr>
<td>USGS Flume A</td>
<td>12.6</td>
<td>50</td>
</tr>
<tr>
<td>USGS Flume B</td>
<td>15.5</td>
<td>40</td>
</tr>
<tr>
<td>Shum Wan Landslide</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Fei Tsui Road Landslide</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Sham Tseng San Tsuen Debris Flow</td>
<td>20</td>
<td>35</td>
</tr>
<tr>
<td>1990 Tsing Shan Debris Flow</td>
<td>20</td>
<td>35</td>
</tr>
<tr>
<td>2000 Tsing Shan Debris Flow</td>
<td>20</td>
<td>30</td>
</tr>
<tr>
<td>Tate's Cairn Landslide</td>
<td>20</td>
<td>35</td>
</tr>
<tr>
<td>Lo Wai Debris Flood</td>
<td>20</td>
<td>30</td>
</tr>
</tbody>
</table>

Note: $\phi$ – Angle of Friction

Group A – Dam Break Scenario

The dam break scenario is based on an analytical solution by Mangeney et al. (2000). The case basically considers a sudden release of the vertical support of a sand mass and the analytical solution presented by Mangeney calculates the profile of the soil at various times. The soil is assumed to behave like a Coulomb material with an internal friction of $\phi$, but hydrostatic internal stresses. The present case was analyzed using an internal friction angle equals to zero.

The results of the analysis using the current model are shown in Figure 1. Figure 1 shows the comparison of the debris profile based on the analytical solution and the computer modeling at different time steps. Figure 2 shows detailed comparisons of the debris profile at various time steps. It is seen that the computer simulation is able to simulate the dam break scenario and the soil profile matches reasonably well with the analytical solution. However, there is a significant difference in boundary conditions between the two cases. The computer model has a finite mass of soils while the analytical solution provides continuous flowing of material and maintains the initial height of soil at some distance from the initial face. Due to this difference in boundary condition, the amount of material available for flow is less in the computer model and the calculated soil profile is in general, lower than the analytical solution.

Figure 3 compares the displacement for the front of the debris at various times compared to the analytical solution. It is seen that the calculated displacement of the front is less than the analytical solution. This is believed to be due to the same boundary effect as discussed above. Also the analytical solution has zero internal forces which is different from the numerical solution. The analytical solution has higher soil profiles, thus resulting in a faster flow down the slope. The velocity of the front is increasing and it is expected that the front will not come to rest until the entire debris slides down the slope. This is because the present slope is 30° and the material has internal and basal friction of 25°. Figure 4 shows the velocity of the front at different horizontal distances. It is seen that after the initial acceleration of the soil mass, the front is moving down the slope at a constant velocity. Therefore, there is no resting position for this case.
Figure 1: Comparison of calculated soil profile based on analytical solution and computer simulation for dam break scenario

Figure 2: Comparison of calculated soil profile based on analytical solution and computer simulation for dam break scenario at various time steps
Figure 2 (Con’t): Comparison of calculated soil profile based on analytical solution and computer simulation for dam break scenario at various time steps.
Figure 3: Comparison of calculated debris front location based on analytical solution by Mangeney et al. (2000) and computer simulation for dam break scenario at various time steps

Figure 4: Calculated velocity of the front of debris versus horizontal distance for Dam Break Scenario
Group A – Sand Flow Experiment
Sand flow experiments were carried out by Manzella at the Swiss Federal Institute of Technology, Lausanne, EPFL. The experiment used a fine sand, called Hostun sand, with an internal friction angle of 34° and basal friction of 32°. During the experiment, the materials slid down a plane and then took a turn onto another plane before it came to rest. This is a three dimensional case and the current two dimensional model can only model planar flow. Therefore, sections in the middle of the flowing mass were chosen for modeling. The initial slope has an angle of 37.5° which is connected by a slope of 22°. Since the material has an internal friction of 34° with a basal friction of 32°, the initial slope is steeper than the basal friction of the material where sliding began. The second part of the slope has a much lower slope angle than the basal friction, therefore, the material came to rest before reaching the bottom of the sliding surface. Figure 5 shows the calculated sand profile at various time steps along the sloping plane using the material properties stated here. Figure 6 shows details of the calculated soil profiles and compares with the final shape of the soil deposit. The entire flow process is completed in about 5 seconds in the model. The final shape of the sand compares reasonably well with the experimental observations. The model has a more gradual soil profile in comparison with the measured one. The velocities for the front of the sand at various horizontal distances are shown in Figure 7. The front has a maximum velocity of about 1.7 m/sec and it came to rest at about 2.7 m from its initial position.

Figure 5: Calculated sand profiles at various time intervals of the Sand Flow Experiment
Figure 6: Final calculated and observed sand profile of Sand Flow Experiment

Figure 7: Calculated velocity of the front of sand at slope distance of the Sand Flow Experiment

**Group A – USGS Flume Experiment**

The information for this case is documented in a paper by Iverson et al. (2004). It is a small scale flume experiment which used two types of sand presented as Flume A and Flume B.
experiments. In Flume A, the sand is angular with diameters of grain size from 0.5 to 1 mm and internal friction of around 44°. The material slid down a plane of about 30° that is made of two materials: Formica and urethane with basal friction of 23.5° and 20° respectively. Only the case sliding on Formica is modeled in this study. Flume B uses a rounded sand with an internal friction of 40° and basal friction of 25.6° over Formica. The sand has a smaller grain size of 0.25 to 0.5 mm.

**Flume A**
The soil in the experiment has an initial height of 4.35 cm with an initial volume of 308 cm³. Figure 8 shows the calculated profile for Flume A at various times. The total modeling time was 1.5 sec. As seen in Table 1, the internal friction and basal friction for the material were reported to be 44° and 24°. However, the model did not get a good match with the final profile of the sand. The best fit is based on an internal friction and basal friction of 50° and 26°, respectively. The calculated velocity is shown in Figure 9. The maximum velocity is calculated to be around 1.2 m/sec when the front reached a distance of around 0.2 m down the plane. Figure 10 shows the comparison between the calculated displacement of the front and the experimental measurements. It is seen in this figure that the calculated front displacement agrees very well with the measurements. A dam break analysis based on the analytical solution by Mangeney et al. (2000) was also carried out. It is noted that there is a significant difference between the observed front displacement and the calculated front displacement using the analytical solution. As well, the front displacement based on the dam break analysis is increasing, indicating that the velocity of the sand is increasing. The analytical solution will not predict an arrest of the material since the slope of the sliding plane is constant and is steeper than the basal friction of the material.

![Flume A](image)

**Figure 8: Calculated sand profile of USGS Flume A Experiment**
Figure 9: Calculated velocity of the front of sand at in USGS Flume A Experiment

Figure 10: Comparison of measured and calculated front displacement of sand using Dam Break Model and computer model

**Flume B**

A similar simulation was carried out for Flume B. The results are shown in Figures 11 and 12. The material parameters used that provide the best fit of the experimental results are the values
measured for this material; that is, an internal friction of 44° and a basal friction of 26°. Although the calculated final soil profile is lower than the observed height, a good match is obtained for the front displacement as shown in Figure 13. Again, the dam break analytical model was used to calculate the displacement of the front which results in a significant over-prediction of the displacement.

Figure 11: Calculated sand profile of USGS Flume B Experiment

Figure 12: Calculated velocity of the front of sand at in USGS Flume B Experiment
Group B – Shum Wan Landslide, Hong Kong
The Shum Wan landslide represents a case of planar sliding with limited mobility, in comparison with other cases in which the debris travelled a much longer distance from its source. More detail information about this case can be found in GEO (2006a).

The results of the current modeling are shown in Figures 14 and 15. Figure 14 shows the progression of the landslide at different times. Although the slide was simulated in 20 seconds, movement was basically over in 16 seconds or less. The calculated final profile in Figure 14 shows a reasonable agreement with field observations. There is some over-prediction of the material lying on the flat ground and under-prediction of the material on the slope. The internal and basal frictions used for the best fit are 30° and 16°. The velocity of the front is shown in Figure 15. This figure shows that the front of the debris travelled approximately 120 m horizontally from its initial position with a maximum velocity of about 13 m/sec.

Parametric variation of the soil friction was carried out to study the effects on the soil profile and runout velocities. The results are summarized in Table 2. Both the internal and basal friction was varied. The case that best fit the observed profile provides an internal and basal friction of 30° and 16° respectively.
Figure 14: Calculated soil profiles at various time intervals of the Shum Wan Landslide

Figure 15: Calculated velocity of the front of the debris versus horizontal distance of the Shum Wan Landslide
Table 2: Material parameters used in modeling the Shum Wan Landslide

<table>
<thead>
<tr>
<th>Shum Wan Landslide Cases</th>
<th>$\phi^\circ$ (Internal)</th>
<th>$\phi^\circ$ (Basal)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>35</td>
<td>22</td>
<td>Base case</td>
</tr>
<tr>
<td>Case 2</td>
<td>30</td>
<td>22</td>
<td>Not much effect</td>
</tr>
<tr>
<td>Case 3</td>
<td>35</td>
<td>16</td>
<td>Move to lower slope, a little overshoot</td>
</tr>
<tr>
<td>Case 4</td>
<td>35</td>
<td>18</td>
<td>Better match - too thin at the top of the final debris</td>
</tr>
<tr>
<td>Case 5</td>
<td>35</td>
<td>20</td>
<td>Too much material at the top of the final debris</td>
</tr>
<tr>
<td>Case 6*</td>
<td>30</td>
<td>16</td>
<td>Toes moved closer, not much effect on the body</td>
</tr>
</tbody>
</table>

* Case 6 represents the best fit of debris profile compared with field observations

**Group B – Fei Tsui Road Landslide, Hong Kong**

A brief description of the Fei Tsui road landslide can be found in GEO (2006b). The failure involved a cut slope of 14,000 m$^3$ of material. It is another case in which the material had basically moved to the bottom of the slope without travelling a significant distance from its source. The modeling results are shown in Figure 16 and the final calculated debris profiles at various time intervals are shown in Figure 17. The calculation was based on internal and basal frictions of 30$^\circ$. Figure 17 shows that the calculated final profile agrees reasonably well with the field observation. The motion of the landslide was simulated in 8 seconds when it came to rest at a horizontal distance of about 40 m from its initial position. The movement suggests that failure initiated at the toe of the slope resulted in subsequent movement of the material higher up the slope. The velocity of the front reached a maximum value of about 11 m/sec in the early stages of sliding as shown in Figure 18. There are various peaks of velocities profile, indicating periods of acceleration and deceleration at the front during the course of movement.

Figure 16: Calculated debris profile of Fei Tsui Road Landslide
Figure 17: Comparison of calculated and observed debris profile of Fei Tsui Road landslide

Figure 18: Calculated velocity of the front of the debris versus horizontal distance of the Fei Tsui Road Landslide
Parametric variation of the soil friction was carried out in this case. The results are summarized in Table 3. The case that provides the best fit for field observation has an internal and basal friction of 30°.

Table 3: Material parameters used in modeling the Fei Tsui Road Landslide

<table>
<thead>
<tr>
<th>Fei Tsui Road Landslide Cases</th>
<th>(Internal)</th>
<th>(Basal)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>35</td>
<td>20</td>
<td>Front move too far - about 95 m</td>
</tr>
<tr>
<td>Case 2</td>
<td>35</td>
<td>30</td>
<td>Front move closer - about 80 m</td>
</tr>
<tr>
<td>Case 3*</td>
<td>30</td>
<td>30</td>
<td>Front about right, not as far - 78 m field is 70 m</td>
</tr>
<tr>
<td>Case 4</td>
<td>30</td>
<td>30</td>
<td>Front did not move compare to Case 3, more budging of the mass</td>
</tr>
</tbody>
</table>

* Case 3 represents the best fit of debris profile compared with field observations

**Group C – Lo Wai Debris Flood, Hong Kong**

The final case presented here is the Lo Wai Debris flood. On August 20, 2005, the Lo Wai debris flood was probably triggered by two major landslides north of Lo Wai in the Shing Mun catchwater area. The material was highly mobile and the flow involved a considerable amount of water. Detailed description of the event is given in MGSL (2006).

The results of the present study are shown in Figures 19 to 21. The calculated soil profiles suggested that the debris is spread partly in the more gentle upper part of the slope and some in the steeper lower part of the slope. The total modeling time took 40 seconds when the velocity of the front is small, but not zero as shown in Figure 21. This figure also shows a maximum velocity for the front of over 16 m/sec which occurred at a horizontal distance of about 220 m from its initial position.

![Figure 19: Calculated debris profile of Lo Wai Debris Flood](image-url)
(a) Lo Wai Debris Flood - Calculated profile at 8 sec

(b) Lo Wai Debris Flood - Calculated profile at 16 sec

Figure 20: Calculated debris profile of Lo Wai Debris Flood at various time steps
(c) Lo Wai Debris Flood - Calculated profile at 24 sec

(d) Lo Wai Debris Flood - Calculated profile at 32 sec

Figure 20 (Con’t): Calculated debris profile of Lo Wai Debris Flood at various time steps
(e) Lo Wai Debris Flood - Calculated profile at 40 sec

Figure 20 (Con’t): Calculated debris profile of Lo Wai Debris Flood at various time steps

![Calculated profile of Lo Wai Debris Flood at various time steps](image)

**CONCLUSIONS**
This paper presents the modeling results of soil movement for 10 cases of landslides and debris flow experiments. The model used in this study is based on a depth averaging model with modifications to include the effects of internal energy dissipation. The inclusion of the energy
dissipation has an effect on the boundary stress between slices. The material behavior was modeled using the Mohr Coulomb model.

There are 3 cases in Group A which consists of one analytical case, one experimental study by EPFL and one small scale experimental study by USGS. Comparison of the results between the calculated and observed soil movement shows that the present model is able to analyze soil movement with reasonable accuracy. There is larger discrepancy when comparing with the analytical solution which neglects internal energy dissipation. In most cases, the measured soil parameters were used, with the exception of one case where the soil parameters were changed slightly to obtain the best results.

Four cases were analyzed in Group B, Shum Wan Landslide, Fei Tsui Road Landslide, Sham Tseng San Tsuen Debris Flow, and the 1990 Tsing Shan Debris Flow. The calculated and observed final debris profiles are compared in these cases. The present model was able to model movement process and the final calculated soil profiles compared well with field observations. The model also provides the velocity profile during the course of movement for these cases.

Only 3 out of the 5 cases in Group C were analysed. The cases that were analyzed are the 2000 Tsing Shan Debris Flow, Tate's Cairn Landslide, and Lo Wai Debris Flood. Frank Slide, Canada and the Thurwieser Rock Avalanche, Italy, are not studied since they are rock slides. The cases analyzed involved rapid mass movement which travelled a considerably farther distance than the previous cases. The calculated velocities are around 16 m/sec, but as high as 34 m/sec. The calculated soil profile also spread along the flow path and in the case of the 2000 Tsing Shan Debris Flow, it was separated into two streams.

Although the present model has its limitations, such as its inability to add and remove material during the flow process to model entrainment and deposition, it is able to calculate the movement of soils in a variety of cases with reasonable accuracy.

REFERENCES
Hong Kong.

**ACKNOWLEDGEMENTS**
This research is supported by the Natural Science and Engineering Council of Canada.
LANDSLIDE MOBILITY ANALYSIS USING MADFLOW

Joanna H. Chen
Golder Associates Ltd, Calgary, Alberta, Canada

C. F. Lee
Department of Civil Engineering
The University of Hong Kong, Hong Kong, China

Abstract: MADflow is a numerical simulation tool for mobility analysis of fast landslides, debris flows and avalanches in multiple rheological regions. Four cases, the deflected flow experiment, the Thurwieser rock avalanche in Italy and two landslide cases in Hong Kong, were analyzed using MADflow. Reasonable results of the modelling show its flexibility and efficiency. This technique is of value in assessment of case histories and potential debris flow/landslide hazards and providing design parameters for both short- and long-term conceptual mitigation strategies.

INTRODUCTION

MADflow was developed by Dr. H. Chen during her PhD study of Mechanism and Modelling of Landslides at the University of Hong Kong, supervised by Prof. C. F. Lee. Formulated with the finite element method in the Lagrangian reference frame under vertical rectangular coordinates system, MADflow is a numerical simulation tool for mobility analysis of fast landslides, debris flows and avalanches in multiple rheological regions. It solves a depth-averaged (quasi-three dimensional) dynamical model in a time marching manner based on two fundamental assumptions: (1) typical flow depth is much smaller than the length scale in the flow direction, and (2) small topographic curvatures. MADflow was first summarized and published in Chen and Lee (2000) and further extended to consider basal erosion effect during debris runout in Chen et al. (2006). It has been applied to slope failure mobility analysis of various case histories (e.g. Chen & Lee 2002, 2003; Crosta et al. 2004, 2005, 2006).

For the 2007 Hong Kong Benchmarking Exercise, four cases were analyzed using MADflow:

(1) Deflected flow experiment by Swiss Federal Institute of Technology;
(2) Thurwieser rock avalanche, Italy;
(3) Shum Wan Road landslide, Hong Kong; and
(4) Fei Tsui Road landslide, Hong Kong.

MODEL DESCRIPTION

To model movement of vigorous debris flows, landslides, rock- and snow-avalanches, we employ depth-integrated laws of mass and momentum conservation. Debris particles are assumed in close contact and moving relatively to one another during runout. In a rectangular Cartesian coordinates system $x = (x, y, z)^T$ with $z^-$ pointing upward opposite to the direction of gravity, the motion of a vertical debris column with volume $V$ and horizontal projection area $A$ is traced in the Lagrangian frame of reference for its trajectory $x(t)$. Such a debris
column satisfies the fundamental principles of mass and momentum conservation in the context of continuum mechanics:

\[
\frac{d}{dt} \int_V \rho \, dV = Q \tag{1}
\]

\[
\frac{d}{dt} \int_V \rho \, u \, dV = \int_V F \, dV + \int_S T \, dS \tag{2}
\]

where \( \frac{d}{dt} \) is the material derivative with \( u = (u, v, w)^T \) being the velocity vector. \( \rho \) is the bulk density of the moving material and is assumed constant. \( F \) is the volumetric force density which consists of \( F = N + P + G \) with the basal normal force \( N \), the net inter-column force \( P \) and the gravity force \( G \). \( T \) is the shear traction force acting on the bounding surface \( S(t) \) of the column, which is a function of different rheological constitutive relationships. \( Q \) is the mass flux rate of the eroded material through the contact surface with the bed \((Q = 0 \text{ for non-erosion})\).

Let \( b(x, y, t) - z = 0 \) and \( f(x, y, t) - z = 0 \) be the basal and free-surface elevation functions, respectively. The vertical column height is thus \( h = f - b \). Defining the gradient operator \( \nabla = (\partial / \partial x, \partial / \partial y, -1) \), the unit normal vector of the bed is:

\[
\mathbf{n} = \frac{\nabla b}{|\nabla b|} = \frac{1}{q} \left( \frac{\partial b}{\partial x}, \frac{\partial b}{\partial y}, -1 \right)^T \tag{3}
\]

where \( q = [(\partial b / \partial x)^2 + (\partial b / \partial y)^2 + 1]^{1/2} \) is geometrically the inclination between the tangent plane of \( b \) and the horizontal \( x-y \) plane. By convention of the right-hand system, \( \mathbf{n} \) is pointing outward from the moving column.

The material derivative on the left-hand side of momentum Eq. [2] can be expressed into two parts:

\[
\frac{d}{dt} \int_V \rho \, u \, dV = \int_V \frac{d(\rho \, u)}{dt} \, dV + \int_S \rho \, u \,(u \cdot \mathbf{n}) \, dS \tag{4}
\]

where the last term is the momentum flux accompanying the change of volume that joins/leaves the original system. We assume that the erosion/deposition occurs only through the column contact area, \( A_c \), with the bed. \( A_c \) can be readily computed from its horizontal projection, \( A \), via \( A = (A_c, \mathbf{n}) \, z = q^{-1}A_c \) with \( z = (0, 0, -1)^T \). By virtue of \( dS = dA_c = q \, dA \), Eq. [4] becomes:

\[
\frac{d}{dt} \int_V \rho \, u \, dV = \int_V \frac{d(\rho \, u)}{dt} \, dV - \int_A \rho_b \, u_b \, w_b \, dA \tag{5}
\]

where \( u_b = (0, 0, w_b) \) is the actual velocity at the bed with \( w_b = db/dt \) being the speed of bed lowering \((w_b < 0)\) / elevating \((w_b > 0)\) in the vertical direction. The horizontal component of \( u_b \) vanished due to the non-slip boundary condition by which \( b \) is defined. \( \rho_b \) is the bulk density of the bed material. Here, we deal with the bed material that have the same bulk density as the sliding material, \( \rho_b = \rho \). However, it is possible that the entrapped material could have
different rheological properties from the original source.

For vigorous debris flows, landslides, rock- and snow-avalanches, the common geometrical property is that the spreading is more predominant than the depth in scale, and translation is more significant than rotation in movement. It is thus justifiable to assume that the debris column remains vertical during displacement so that \( dV = hdA \), and most importantly, any depth-averaged variable, in the sense of:

\[
\overline{\Phi} h = \int_b^f \Phi dz
\]  

is reasonably representative to the simple variation of \( \Phi = \{ \rho \mathbf{u}; F \} \) along the vertical direction. Using Leibniz’ theorem to the depth integration, one also has

\[
\frac{d(\overline{\Phi} h)}{dt} = \frac{d}{dt} \int_b^f \Phi dz = \int_b^f \frac{d\Phi}{dt} dz + \Phi \frac{df}{dt} - \Phi_b \frac{db}{dt}
\]  

[6b]

where \( \Phi_f \) and \( \Phi_b \) are the values of \( \Phi \) taken at the free surface and the bed, respectively. Subsequently, [5] reads (with \( \Phi = \rho \mathbf{u} \)):

\[
\frac{d}{dt} \int_V \rho \mathbf{u} dV = \int_A \int_b^f \frac{d(\rho \mathbf{u})}{dt} dz dA - \int_A \rho \mathbf{u}_b w_b dA = \int_A \left( \frac{d(\rho \overline{\mathbf{u}} h)}{dt} - \rho \mathbf{u}_f \frac{df}{dt} + \rho \mathbf{u}_b \frac{db}{dt} - \rho \mathbf{u}_b w_b \right) dA
\]  

[7]

where \( \mathbf{u}_f \) is the actual velocity at the free surface. As we do not consider any mass exchange at the free surface, \( df/df = 0 \). The momentum equation in \( x\)-\( y \) plane simplifies as:

\[
\int_A \frac{d(\rho \overline{\mathbf{u}} h)}{dt} dA = \int_A \overline{\mathbf{F}} h dA + \int_A \mathbf{T} dA
\]  

[8]

For a typical debris column, we assume that the mass flux rate of erosion is proportional to the surface area \( A_c \) affected by erosion and the magnitude of the averaged debris velocity \( | \mathbf{u} | \), so that:

\[
Q = \int_S \rho (\overline{\mathbf{u}}_k \cdot \mathbf{n}) dS = \int_A E \rho | \overline{\mathbf{u}} | q dA
\]  

[9]

where \( E \) is the so-called yield rate, positive for entrainment and negative for deposition. \( E \) is dimensionless and is defined as that \( E \) units of volume of material is to be eroded via a unit contact area as the debris travels a unit length of its path. Intuitively, it may also be interpreted as that a finite mass with a unit contact area and traveling a unit length in the course of sliding will result in a lowered basal surface by \( E \) unit height (entrainment depth).

The reader is referred to Chen et al. (2006) for detail of force analysis, finite element idealization, and discussion of rheological constitutive relationship and parameter selections.
CASE STUDIES

Deflected Flow Experiment
The deflected flow experiment was conducted by Rock Mechanics Laboratory of Swiss Federal Institute of Technology Lausanne EPFL. In their experiment, the release box has the dimensions 20 cm by 40 cm by 65 cm. About 30,000 cm$^3$ fine sand was stored in the box and was then released by the sudden removal of the down slope wall of the box. The sand fall height is 100 cm, measured from the opening gate as shown in the annex drawing to the package. A contour of the experimental path and the final deposit was provided. The sand has an internal friction angle ($\phi$) of 34° (repose angle). The basal friction angle ($\delta$) between the fine sand and the base material (forex) is 32°, estimated from a tilting test.

Back-analysis of the deflected flow experimental test was conducted using MADflow. The discretized sand volume is the same as the experiment. Optimized correlations between the experimental observations and numerical simulation are given by:

- Rheological constitutive relationship: friction model;
- Internal friction angle ($\phi$) of the released sand: 34°;
- Apparent friction angle ($\phi_b$): 31°;

<table>
<thead>
<tr>
<th>$k_\xi^a$</th>
<th>$k_\eta^p$</th>
<th>$k_\xi^a (k_\xi^a)$</th>
<th>$k_\eta^p (k_\xi^a)$</th>
<th>$k_\eta^p (k_\eta^p)$</th>
<th>$k_\eta^p (k_\eta^p)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.28</td>
<td>3.54</td>
<td>0.28</td>
<td>1.0</td>
<td>1.0</td>
<td>3.54</td>
</tr>
</tbody>
</table>

Figure 1 shows selective images of the calculated sequences at instants of 0.42, 0.56, 1.12 and 1.54 seconds, compared with corresponding experimental observations at the same instants. After the sand is released from the gate, it accelerates downward rapidly on the inclined flat plane without side constrained, spreading laterally and forming a teardrop as it goes. As the leading edge passes the transition zone and enters the deflected board, the front rapidly changes to the main descending direction of the deflected board. The profile reshapes from a teardrop to a boot with a pointed front and a blunt tail. As the sand is coming to rest, the tail pushes forward and the profile resumes a bullet form. The simulation shows satisfactory agreement with the experimental observations. The entire runout duration takes about 3.36 second for the final deposition, which also agrees with the experiments.

The corresponding velocity vectors are shown in Figure 2. Maximum and mean velocities during runout are summarized in Figure 3.

Figures 4 and 5 show the calculated final deposition contours, compared with measurements provided by the Lausanne group. The deposition profile correlates well with the measurements. However, the maximum deposit thickness approximates 8.5 cm, which is somehow lower than the measured thickness of 10 cm. This is presumably attributed to the slower tail movement in the simulation. It is worthwhile mentioning that the initial shape of the material is slightly different from that of the experiment due to the use of the vertical coordinate system. This could have played a minor role in the final deposition. After all, the traveling distance is not sufficiently long in this flow event.
Mesh refinement test was conducted for this case by element quartering. The result remains essentially the same, with a much better maximum deposition height of 9.6 cm closer to the measured thickness of 10 cm, but minor over-prediction at the front. Shock waves, which are more apparent on the refined mesh, develop as shown in the accompanying animation.

Figure 1: Calculated sequence at $t = 0.42, 0.56, 1.12$ and $1.54$ seconds, compared with corresponding experimental observations
Figure 1 (Con’t): Calculated sequence at $t = 0.42$, $0.56$, $1.12$ and $1.54$ seconds, compared with corresponding experimental observations.

Figure 2: Calculated velocity vectors at $t = 0.42$, $0.56$, $1.12$ and $1.54$ seconds.
Figure 3: Summary of maximum and mean velocities during runout

Figure 4: Calculated final deposition contours (lower left - coarse grid; lower right – fine grid), compared with experimental measurement (upper).
Figure 5: Calculated final deposition depth distribution (fine grid)

**Thurwieser Rock Avalanche, Italy**
The Thurwieser rock avalanche occurred on September 18, 2004 in central Italian Alps. The avalanche mainly consisted of dolostone and traveled over 2.9 km from its source zone to along the Marè valley, involving about 2.2 Mm$^3$ of rock. The major duration of the avalanche propagation took about 75 – 80 seconds after the rock detached from the source zone scar. The detachment scar was approximately empty after about 1 min 30 seconds.

Interpreted from the available photos, the rock detachment area (source zone) approximates at elevation 3,657.6m above sea level (a.s.l.). A relatively flat area is presented at elevations 3,000 – 2,900m a.s.l. Almost no deposition was observed along the glacier. Most of the tributary valleys located in the upper parts were still partially covered by glacial ice.

After reaching the limit of the glacial cirque, some rock debris fell down the outcropping rock; while the majority of rock debris moved along the narrow valley which seemed to have been confined by the lateral moraines and the outcropping rock. The rock is outcropping from elevations 2900 m a.s.l. to 2600m a.s.l. where glacial deposits and exposed rock start. The estimated velocity of the rock debris falling down the outcropping rock ranged between 42 and 53 m/s (149.6 – 190 km/hr), and increased to 57 m/s (206 km/hr) after the avalanche fell down the outcropping rock and reached the glacier.

Reaching the lower end of the outcropping rock at elevation 2600m a.s.l., the avalanche moved with estimated velocity of 46 – 38 m/s (166 – 138 km/hr) at elevations 2550 – 2450m a.s.l. The valley floor was covered by glacial deposits and exposed rock.

The avalanche turned progressively to its right hand flank at elevation 2400 – 2200m a.s.l., where the topography appeared to have been controlled by a small ridge consisting of glacial deposits. More or less a turn and a slight run-up were observed. The majority of the avalanche seemed to have stopped traveling shortly after the significant velocity of 17 m/s.
Based on the above and the brief descriptions provided by the University of Milan, it is understood that the entire runout consists of three major stages: rock detachment, majority of rock debris moving along the narrow valley confined by the lateral moraines and the outcropping rock, and moving along the valley floor. Small amount of debris experienced rock fall from the outcropping rock (which ends at 2600m a.s.l.). It is also noted that the upper part of the valley floor was covered by glacial ice, following with glacial deposits and exposed rock in the lower portion of the slope. Therefore, due to the basal conditions, two rheological regions (1, 2) are partitioned by where the outcropping rock ends.

Back-analysis of the Thurwieser rock avalanche was conducted using MADflow. The discretized detachment volume is 1.84 Mm$^3$ without accounting for fragmentation of about 20%. Optimized correlations between the field observations and numerical simulations are given by:

- Rheological constitutive relationship: Voellmy model;
- Internal friction angle ($\phi$) of rock debris: 38°;
- Turbulent coefficient: 500 m/s$^2$;
- Two rheological regions: Region 1 with $\mu = 0.18$ ($\tan 10^\circ$) and Region 2 (where elevations below 2600m a.s.l.) with $\mu = 0.36$ ($\tan 20^\circ$).

Table 2: Coefficients of active and passive earth pressure

<table>
<thead>
<tr>
<th>$k_a^p$</th>
<th>$k_p^a$</th>
<th>$k_a^p (k_p^a)$</th>
<th>$k_p^a (k_a^p)$</th>
<th>$k_a^p (k_a^p)$</th>
<th>$k_p^a (k_p^a)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.24</td>
<td>4.20</td>
<td>0.24</td>
<td>1.0</td>
<td>1.0</td>
<td>4.20</td>
</tr>
</tbody>
</table>

Figure 6 shows selective images of flow sequence at instants of 13, 44, 86, 240 and 300 seconds. After departing from the source zone at elevation 3657.6m a.s.l., the avalanche moves along the glacier down to a relatively flat area at elevations 3000–2900m a.s.l. The calculated maximum velocity (front) approximates 63 m/s at $t = 13s$ shortly before reaching the flat area of the glacial cirque.

After reaching the limit of the glacial cirque, the program successfully simulated the thin, wide spread rock debris on the morainal slopes below the glacier. The maximum simulated velocities of debris sliding on the upper and lower half outcropping rock approximate 32 m/s and 37 m/s (stages 2 and 3) respectively. The majority of rock debris moves along the narrow valley. When the avalanche moves to the end of the outcropping rock and reaches the glacier, the calculated front velocity approximates 35 m/s at $t = 44s$ (stage 4). The field estimated velocities seem to have represented the rock fall velocities (stages 2, 3 and 4). The comparison here might be meaningless since the steep slope of the outcropping rock has made the assumption of the depth-averaging model invalid.

From the lower end of the outcropping rock to elevation 2450m a.s.l. along the valley floor, the calculated velocity is 33 m/s at $t = 71s$, which is close to the field estimates of about 38 m/s (stage 5).

At $t = 86s$, the tail of simulated avalanche is emptying from the detachment scar, which is consistent with the field observation of about 1 minute and 30 seconds. It is also at this moment when the avalanche front touches the valley floor with velocity of 25 m/s.
At $t = 185$ s (not shown in the figure), some rock debris close to the tail is overflowing the left side of the outcropping rock, which is consistent with the field observations (from photos).

Afterwards, the moving velocity gradually reduces to 16 m/s at $t = 240$ s. The front is close to the elevation 2260 m a.s.l. The simulated velocity is in good agreement with the field observations of 17 m/s (stage 6). The simulated avalanche turns progressively to its right hand flank with a slight run-up, which also correlates well with the field observations.

From $t = 240$ s to $t = 300$ s, the runout velocity rapidly reduces when it moves along the valley floor, as shown in Figure 8. At $t = 300$ s, the simulated average velocity is around 0.4 m/s, showing that the overall avalanche has settled down although about 1 – 2 m/s velocity exists caused by the tail pushing forward. The deposits accumulate up to about 19 m in thickness.

The simulated avalanche then stops around elevation 2250 m a.s.l.. The black solid lines in Figure 6 shows the runout hazard zone, which correlates well with the field observations. Corresponding velocity vectors are shown in Figure 7. The maximum and mean velocities during runout are summarized in Figure 8.

Figure 6: Calculated sequence at $t = 13$, 44, 71, 86, 240 and 300 seconds (flooded contours show the corresponding depth)
Figure 6 (Con’t): Calculated sequence at $t = 13, 44, 71, 86, 240$ and $300$ seconds (flooded contours show the corresponding depth)

Figure 7: Calculated velocity at $t = 13, 44, 71, 86, 240$ and $300$ seconds
Figure 7 (Con’t): Calculated velocity at $t = 13, 44, 71, 86, 240$ and $300$ seconds

![Figure 7](image)

Figure 8: Summary of maximum and mean velocities during runout

**Shum Wan Road Landslide, Hong Kong**

The Shum Wan Road Landslide took place on 13 August 1995 in the southern part of the Hong Kong Island. A heavy rainstorm preceded the landslide. The landslide collapsed from the Nam Long Shan Road, crossed the Shum Wan Road, damaged three shipyards and a factory near the seafront, and resulted in two fatalities and injury to five. The landslide released about 26,000 m$^3$ of soil and rock and it is the largest landslide in Hong Kong over the last twenty years.

The released debris comprised mainly a very soft or loose fluvial deposit of clay, silt, sand, gravel and some cobbles and boulders. Prior to the subject landslide, an area of fill, up to about 5 m thick estimated from aerial photographs, covered the upper part of the hillside. The fill generally had a degree of compaction less than 80% of the Standard Proctor maximum dry density. Soil tests indicate that the frictional strength of the clay seam with slickensiding
is $\phi' = 22^\circ$ and $c' = 0$ kPa. It has been concluded that the presence of weak layers in the ground and the ingress of water during prolonged heavy rainfall principally contributed to the failure (GEO 1996a).

Back-analysis of the Shum Wan Road landslide was conducted using MADflow (Chen & Lee 2000). The simulated volume of the detached mass in the source zone is 25,384 m$^3$. Optimized correlations between the field observations and numerical simulation are given by:

- Rheological constitutive relationship: friction model;
- Apparent friction angle ($\phi_b$): 20$^\circ$.

### Table 3: Coefficients of active and passive earth pressure

<table>
<thead>
<tr>
<th>$k_x^a$</th>
<th>$k_z^p$</th>
<th>$k_x^p(k_z^a)$</th>
<th>$k_x^a(k_z^p)$</th>
<th>$k_y^a(k_z^p)$</th>
<th>$k_y^p(k_z^a)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.8</td>
<td>2.5</td>
<td>0.8</td>
<td>1.0</td>
<td>1.0</td>
<td>2.5</td>
</tr>
</tbody>
</table>

![Figure 9: Calculated sequences (flooded contours show the corresponding depth)](image-url)
Figure 9 shows the selected sequences of the simulated depth contour distributions of the sliding debris at instants of 0, 5, 9, 15, 21 and 33 seconds, demonstrating that the spoon-shaped patch moves continuously and grows larger progressively during the movement. Mean average velocities are also shown at these instants. The maximum local velocity, about 14.40 m/s, occurs at $t = 9.0s$. The runout lasts about 33s (Figure 10).

![Figure 10: Summary of velocities and footprint area during runout](image)

Figure 10 compares the deposition area with the field observations. The computed scar is slight under-prediction in the sliding direction and minor over-prediction in the transversal direction. It is noted that cobbles, boulders and some clumps of vegetation were observed around the outer edge of the deposition near the seafront. The current calculations use “equivalent fluid” concept and consider the debris as a continuous media. Individual movement of cobbles and boulders might have contributed to the minor discrepancy between the calculations and observations.

**Fei Tsui Road Landslide, Hong Kong**

On the same day as the Shum Wan Road landslide, 13 August 1995, a landslide at Fei Tsui Road in Hong Kong buried the road down the slope by slide debris after a heavy rainstorm,
and caused one fatality and one injured. With an unusual volume of about 14,000 m$^3$, it is reported as the largest fast-moving cut slope failure in Hong Kong over the last decade (GEO 1996b). With an average vertical depth of about 15 m, the landslide formed a scar with a maximum length of nearly 90 m along the Fei Tsui Road and a maximum extent of 33 m from the toe of the slope. The maximum horizontal travel distance of the debris was about 70 m, as measured from the crest of the landslide.

The released debris comprised predominantly coarse gravel-sized to boulder-sized joint-bound blocks of moderately to highly decomposed tuff together with some other materials, such as the asbestos cement pipes, broken pieces of concrete surface channel, broken sections of a masonry wall and concrete blocks. There was a laterally-extensive layer of kaolinite-rich altered tuff dipping approximately to the north at about 10° to 25°. Groundwater table was unlikely to have been above the base of the landslide at the time of failure, and no liquefiable material existed in the sources rock mass or in the path downslope. Direct shear tests found that the average shear strength of the weathered volcanic joints was $\phi' = 35^\circ$ and $c' = 0$. It is concluded by the GEO that the landslide was primarily caused by elevated water pressure in an extensive and weak layer in the slope, following the extremely heavy and prolonged rainfall that preceded the failure.

Back-analysis of the Fei Tsui Road landslide was conducted using MADflow (Chen & Lee 2000). The discretized rock detachment volume is 13,938 m$^3$. Optimized correlations between the field observations and numerical simulations are given by:

- Rheological constitutive relationship: friction model;
- Internal friction angle ($\phi$) of rock debris: $35^\circ$;
- Apparent friction angle ($\phi_b$): $22^\circ$.

<table>
<thead>
<tr>
<th>$k_z^a$</th>
<th>$k_z^p$</th>
<th>$k_\eta^a (k_z^a)$</th>
<th>$k_\eta^p (k_z^a)$</th>
<th>$k_\eta^a (k_z^p)$</th>
<th>$k_\eta^p (k_z^p)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.58</td>
<td>3.38</td>
<td>0.58</td>
<td>1.0</td>
<td>1.0</td>
<td>3.38</td>
</tr>
</tbody>
</table>

Figure 12 shows the selected sequences of the simulated depth contour distributions of the debris at instants of 0, 1.5, 2.5, 4.0, 6.0 and 15 seconds. The variation of the simulated debris depth, the runout area and the final deposition on the playground and the corner of a church are in close agreement with the field observations. Mean average velocities are also shown at these instants. The maximum local velocity of 18.5 m/s occurs at $t = 2.5s$ and the maximum mean velocity of 4.3 m/s occurs at $t = 3.0s$; the runout lasts about 15s (Figure 13).
Figure 12: Calculated sequences (flooded contours show the corresponding depth)
CLOSING REMARKS

MADflow is useful in assessment of present and potential debris flow/landslide hazards and providing design parameters for both short and long-term conceptual mitigation strategies. MADflow uses a very flexible keyword input system and standard data input formats. It follows a multiple region concept that different rheological models and/or parameters can be applied to a single topographic data. The results are exported in an ASCII format native to TECPLIT® (www.tecplot.com), one of the leading post-processors for scientific visualization. MADflow is coded in C++. Its calculation speed is measured about 1.9 milliseconds per node per 1,000 time steps using a Pentium 3.2Ghz computer (single CPU) on Windows® XP, which varies linearly with number of nodes and is independent from topographic resolution. MADflow is very easy to use, although it requires some knowledge of geotechnical and geological engineering and some rudimentary exposure to numerical simulation and data-processing.

Input of the MADflow:

(a) The structured or scattered point coordinates (x, y, h) of the pre-post failure topographic distributions in plain ASCII format. The coordinates may be directly converted from an existing GIS database. MADflow has a built-in converter for the DEM format.

(b) The structured or scattered point coordinates (x, y) of the observed source zone scar, runout scar margin and the deposition margin in plain ASCII format.

(c) Geotechnical properties of the mobilized material

(d) Rheological model and relevant parameters

Output of the MADflow:

(a) Instantaneous velocity and depth variations during runout

(b) Runout hazard zone
REFERENCES


APPROACH TO NUMERICAL MODELLING
OF LONG RUNOUT LANDSLIDES

G. B. Crosta
Dipartimento Scienze Geologiche e Geotecnologie, Milano, Italy

S. Imposimato and D. Roddeman
FEAT, The Netherlands

Abstract: The support of numerical models is determinant to obtain a more sound and complete zonation for long runout mass wastings. Different models have been conceived and developed and are presently available both commercially or for research purposes. In the last few years we implemented a fully 3D finite element code that consents to simulate some of the most complex conditions typical of debris and rock avalanches. Irregular topographies, material variability, sediment erosion and entrainment, effect of water content have been simulated. In this paper we present a series of examples, partially associated with the Benchmarking Exercise carried out for the 2007 Landslide Disaster Forum. We rely on the final report of the Exercise by the Technical Committee for more details.

We introduce the problem, describe the adopted modelling approach, and finally present a series of results concerning both simulations of physical tests and of real case studies for which material is available in the literature (granular step collapse), has been prepared by the Organizing Committee of the Benchmarking Exercise (Frank slide and Shum Wan landslide) or has been directly collected by the authors (Thurwieser rock avalanche).

INTRODUCTION

Rock and debris avalanches are characterized by different behaviours with respect to other landslide types. The evolution of rock and debris avalanches can be strongly influenced by the local conditions at the source area, both in terms of elevation along the slope and in water content. Water content within the landslide mass and the geometry of the initial failure surface can cause an accelerated rock mass opening and fragmentation, and a successive increment in mobility. This flow-like phenomena can travel long distances along irregular topographies, and can overcome or be deflected by topographic obstacles, made of different materials (outcropping rock, thin colluvial deposits, thick loose deposits and ice). During their motion, they can entrain large volumes of these sediments both in a dry state and in a saturated state (Abele 1974, 1997). Dry material has generally the consequence to reduce the total landslide runout, whereas saturated or almost saturated materials can induce more complex consequences. In fact, sudden undrained loading (Hutchinson & Bhandari 1971; Sassa 1988), plowing, impact on loose deposits, liquefaction and/or entrainment of a saturated substrate can enhance mobility causing major consequences on the hazard zonation. The eroded material can be pushed, accreted to the main flow front or entrained at the base and mixed within the moving mass (Abele 1974, 1997; Plafker & Ericksen 1978; Crosta 1992; Crosta et al. 1993; Hungr & Evans 2004).

Because of all these constraints the study of debris and rock avalanches requires a complete analysis by taking into account the initial water content, three dimensional effects connected to failure surface geometry and path topography. Therefore, special attention should be
devoted to hazard zonation of long runout mass wastings.

The support of numerical models is then fundamental to obtain a more rigorous and complete zonation, inclusive of the uncertainty connected to the large natural variability in material properties. Numerical modelling of post-failure motion can be used to estimate the maximum expansion of potential flow-like landslides and to evaluate instantaneous velocities and flow depth, for hazard assessment as well as the design of protective measures.

Different models have been conceived and developed and are presently available both commercially or for research purposes. Many of these present advantages and disadvantages. Most of these approaches start from the observation that the moving layer tends to be extremely elongated, with minimum thickness to length ratio, and the assumption that shearing occurs prevalently at the very base of the material. This approach allows to simplify the problem and, consequently, the construction of one dimensional or two dimensional (quasi three dimensional) models. Different 1D and 2D numerical models have been proposed (Savage & Hutter 1989; Hungr 1995; Chen & Lee 2000; Pitman et al. 2003; Mangeney et al. 2000, 2003; Denlinger & Iverson 2001, 2004, McDougall & Hungr 2004; Chen et al. 2006; Pastor et al. 2002) usually in the form of depth-averaged, Lagrangian numerical methods where the landslide volume can be constant or variable and one or multiple rheologies can be adopted along the path. A drawback of such an approach consists in the difficulty in simulating more complex conditions, like motion of thick masses, flow conditions where a strong vertical component is included, effects of different material conditions and behaviours, dragging and erosion of material located in front or along the path. On the other hand, they present the advantage of requiring few model parameters and of relatively low computational times. In the last few years, we implemented a finite element code (Roddeman 2002, Crosta et al. 2003, 2005 2006 2007) that allows to simulate some of the previously listed problematic conditions.

We present a series of examples, partially associated with the Benchmarking Exercise carried out for the 2007 Landslide Disaster Forum. In the following, we describe the adopted modelling approach, and present a series of results concerning both simulations of physical tests and of real case studies for which material is available in the literature, has been prepared by the Organizing Committee of the Benchmarking Exercise or has been directly collected by the authors.

**MODELLING APPROACH**

In the following, we show the application of a Eulerian-Lagrangian finite element code (Roddeman 2002). We present 2D-3D results obtained by the finite element code, allowing to analyse the motion of the moving landslide mass along very rough topographies (e.g. including sharp geometries as deep and narrow gorges), and on materials with different properties (e.g. hard substrate or erodible soils). Final geometry, mass redistribution, velocities and runup are used to validate the model capabilities.

As said above, sliding and flowing rock and soil mass wastings show very large displacements and deformations. If a traditional Lagrangian finite element method would be used, the finite element (FE) mesh would be subjected to these large displacements and deformations. This would lead quite rapidly to a highly distorted mesh, and the calculated results would become inaccurate. We use a particular type of combined Eulerian-Lagrangian method (Roddeman 2002) that allows large material displacements without distorting the FE
mesh and guarantees accurate calculation results. Besides discretization issues, this influences the material law that should be such that any rigid body rotation should only lead to stress rotation, but not to additional stresses within the flowing mass. The problem consists of the fact that the rigid body rotation component is not uniquely defined as part of an arbitrary deformation pattern and numerous definitions are possible. The flow of landslide material is governed by:

$$\rho \dot{\mathbf{v}}_i = \frac{\partial \sigma_{ij}}{\partial x_j} + g_i$$  \[1\]

where $\rho$ is the density of the material, $\mathbf{v}_i$ is the velocity in $i$-direction ($i$ is 1, 2 or 3), $\sigma_{ij}$ denotes the stress tensor, $x_j$ is the $j$-th space coordinate ($j$ is 1, 2, or 3) and $g_i$ represents the gravity force in $i$-direction ($g_1 = g_2 = 0$, $g_3 = -9.81 \text{ m/s}^2$). The equation holds at the instantaneous location of the sliding mass, and so if at any time the location of the sliding mass and the stresses in the material are known, the material acceleration can be calculated. When the material velocities are derived, the landslide moves to its new location in space. Effective stress changes due to material stiffness result in a fixed frame from the rigid body rotations of the material. Since the rigid body rotations from an arbitrary deformation field are not uniquely defined, a choice has to be made how to model this contribution.

We apply an incrementally objective Lagrangian model, based on a polar decomposition of the incremental deformation tensor. The second contribution comes from straining of the material. For this part we use a Mohr-Coulomb law which states that shear effective stresses cannot exceed the critical value as dictated by the normal stresses in the material. Moreover, the total stress in a calculation with groundwater flow is the sum of effective stress and pore water pressure. The pore water pressure follows from the hydraulic head and the evolution of the hydraulic head if governed by the water storage equation. With this approach, we can describe the large deformations and sliding of landslide material, filled or not filled with groundwater.

Several types of finite elements can be used in the Tochnog FE code (Roddeman 2002). For discretization in time, Euler backward time stepping is applied. We disconnect material displacements from the finite element mesh, state variables are transported through the mesh by a Streamline Upwind Petrov Galerkin method (SUPG).

We apply classical elasto-plasticity to model the non-linear path-dependent behavior of soils including both the falling rock/debris and the erodible material located along the flow path. As a yield rule, the Mohr-Coulomb, Drucker-Prager surfaces have been used and can be coupled. Material properties can be evaluated and attributed by a simple trial and error calibration procedure, or by selecting representative values according to the type of involved rock and/or soil. This is the most general approach because no antecedent event is needed for calibration. Different constitutive laws are available including plastic and hypoplastic laws, hardening anisotropic laws and softening laws.

To perform the simulation, the initial equilibrium stress state is reached through quasi-static time stepping by removing all inertial effects. A pre-defined slip or failure surface, determined from preliminary FE stability calculations, from in-situ evidence like major tension or shear cracks, or from post-event descriptions, can be imposed. The landslide can be triggered by either lowering cohesion in time, or imposing a base acceleration diagram to
simulate seismic triggering. After the landslide is triggered and the mass starts moving, time steps are taken until a solution at rest is obtained.

**Groundwater and Erosion Modelling**
The slide mass can be considered dry or saturated, imposing in this last case a null pressure at elevations above the groundwater table. The assumption, in this set of analyses, consists in allowing drainage only through the upper landslide boundary, and not through the basal surface. Adoption of this numerical modelling approach allows to simulate the contemporaneous presence of different materials with different properties and under different initial conditions. Furthermore, we can simulate erosion/entrainment of material avoiding the use of empirical rules (McDougall & Hungr 2005, Chen et al. 2006) but considering material properties and landslide mass instantaneous characteristics.

**MODEL VALIDATION**

**Granular Step Collapse**
The collapse of a granular step or column is of great interest and has been only recently recognized as an important phenomenon useful for studying transient granular flow conditions. This type of process is quite similar to the dam break problem in fluid mechanics but in this case the material is a granular one. A series of very well detailed experiments is available in the literature (Lajeunesse et al. 2004, 2005; Lube et al. 2004, 2005; Siavoshi & Kudrolli 2005; Balmforth & Kerswell 2005). These tests have been compared both with qualitative and theoretical models, aimed at finding some general scaling laws or at testing some two dimensional depth averaged shallow water models. Other authors used particle mechanics models to simulate these experiments (Staron & Hinch 2005) by assuming friction, and rigid collisions between grains.

The tests described in the literature have been performed with different materials: grit, glass beads (ballotini), polystyrene balls, sand, salt grains, cous-cous grains, rice and sugar, and adopting different geometries: vertical columns or 2D steps in narrow or wide slots (i.e. rectangular channel). Evidently, the different column or step geometry (low or high aspect ratio between height and width), the change in material properties, the use of smooth or rough basal surfaces, the mechanism adopted for releasing material and the width of the slot in 2D tests have an influence on the final results. Nevertheless, the available data are of great interest for model calibration and verification especially considering the type of phenomena we are interested in. The important parameters to be considered for the simulations and the analysis of the results are those listed above and partially described in Figure 1.

We present the results of three simulations (Figures 1, 2, 3 and 4), performed considering an aspect ratio, \( a = h_i/L_i \), of 1, 1.4 and 3.2, with a material considered elasto-plastic and a Mohr Coulomb yield rule (for \( a = 1 \) and 1.4: friction angle \( \phi = \phi_b = 30^\circ \), \( c = 10 \) kPa; for \( a = 3.2 \): \( \phi = \phi_b = 24.8^\circ \), \( c = 1 \) kPa). These values are quite similar to those for the experiments performed with glass ballotini by Lajeunesse et al. (2004, 2005), (1.15 and 3 mm; \( \phi = 22^\circ, \phi_b = 11^\circ \) and 12\(^\circ\)), and by Balmforth & Kerswell (2005); (0.8 and 3 mm; \( \phi = 24.5^\circ \) and 26\(^\circ\), \( \phi_b = 14.75^\circ \) and 15\(^\circ\)).

The geometry of the material profile for an experimental test performed by Lajeunesse et al. (2005) with an aspect ratio equals to 3.2, is shown in Figure 1(b) for different time steps normalized with respect to the square root of the \( H_i/g \) ratio. This can be compared to the ones
obtained numerically and shown in Figure 2(a).

Figure 1: (a) Geometrical features relevant for the description of the granular dam break problem, (b) example of time evolution of the profile of the material for an experimental test by Lajeunesse et al. (2005) with an aspect ratio, $a = 3.2$.

Figures 2 and 3 show the development of the flow since the initial failure. Failure occurs along a clear inclined plane but most of the column is moving in a vertical direction with a free fall behaviour and only the more external and lower half of the material starts moving outward with a horizontal component. These figures clearly show also the progressive increase in size of the static layer, the rotational failure mechanism and the progressive thinning of the moving mass. The progressive migration of the internal interface between static and flowing material for a numerical simulation is shown both in Figures 2(b) and 3.

The general agreement in terms of geometry, pattern of velocity and position at different instants is good and can be compared through some plots of normalized variables (see Figure 4) with experimental data from Lajeunesse et al. (2004, 2005); compare with Figures 10(a) and 10(b) by Lajeunesse et al. (2005), Balmforth & Kerswell (2005) and Lube et al. (2004, 2005).

Figure 2: Evolution of the (a) profile and (b) of the interface between static and moving material for the collapse of a 2D granular step ($a = 3.2$) as from numerical model. Profiles are shown at successive time steps ($\Delta t = 0.05$ s). Figure 2(a) can be compared to Figure 1(b).
Figure 3: Evolution of the collapse of a 2D granular step with $a = 3.2$. Velocity vectors (m/s) are shown at 0.1 sec time interval starting at 0.05 sec. Initial discretization includes 2008 elements. The vertical collapse is evident in the initial time steps, whereas the entire series shows the growth of the static material volume (white area below moving material). Figures (a), (c) and (d) can be compared with Figures 10(a), 8, and 10(b) by Lajeunesse et al. (2005).
Figure 4: Comparison of 2D numerical results with 2D experimental results for different granular materials taken from Lajeunesse et al. (2004, 2005) (Lj et al.), Lube et al. (2004, 2005), and Balmforth & Kerswell (2005) (B&K). L represents the initial length of the column along the channel. The power law relationships proposed in the literature are various (Lajeunesse et al. 2005; Balmforth & Kerswell 2005). In the normalized runout length plot we observe a slight divergence in the trends for the different materials below and above the knee point for an aspect ratio, $a \approx 2$.

Frank Slide
The 1903 Frank Slide, Canada, (McConnell & Brock 1904; Cruden & Krahn 1978) involved some 36 Mm$^3$ of limestone. The rock avalanche detached from the ridge of Turtle Mountain and felt 800 m down to the valley bottom where it destroyed part of the town of Frank causing approximately 70 casualties.

The final deposit was about 1.7 km wide and 2 km long with an average thickness of 18 m. The post-failure topographic data derive from a 1 m DEM obtained by a LiDAR survey (Province of Alberta Geological Survey, AGS) whereas the pre-failure topography was reconstructed by lowering of the valley bottom and reshaping the mountain topography considering an expanded vertical thickness by 20% to account for debris bulking.

No information exists about the movement velocity, however, based on eyewitness interviews, McConnell & Brock (1904) conclude that the “time that elapsed between the first crash and complete rest did not exceed 100 seconds, and may have been somewhat less”. This time interval, the average thickness and the debris limits can be used to validate the model results.

A set of fully 3D analyses has been performed by using a discretization mesh with 60 m long and large cells, and 4m thick. The topography is the one obtained by the distributed dataset.

The landslide material has been considered homogeneous and the spreading surface is differentiated from the initial failure surface from which the material is released. The properties are those reported in the table with constant elastic parameters ($E = $ Young’s modulus $= 1.8E$ Pa, $\nu =$ Poisson ratio $= 0.23$).

In the following figures (Figures 5, 6 and 7), we present some of the results in terms of velocity vectors, the thickness of the material during spreading and the geometry of the moving mass at different time steps during the failure. The moving material comes to rest after 60-90 s. This duration is comparable with the 100 s duration estimated by McConnell
and Brock (1904).

No exact comparison between modelling results and observations has been performed, as well as a complete calibration, because no accumulation limit and map of deposit thickness was made available in digital format. Nevertheless, we superimposed the computed landslide limit to the ortho-rectified image showing a good agreement (Figure 5) between the two limits.

The deposit mass is concentrically distributed and the maximum thickness computed by the model is located in the central part of the deposit and is close to 20-28 m.

The maximum computed velocity reaches values between 72 and 75 m/s. These are prevalently observed in the upper half of the flowing material until the moving mass reaches the flat valley bottom.

Velocity vectors show a clear sub-parallel downslope trend along the main slope and then progressively diverge symmetrically along the flat valley bottom and the opposite rise.

This general pattern seems quite reasonable and comparable with the deposit distribution, its lobate geometry and the flow features (shallow elongated ridges and furrows) that seems recognizable also in the ortho-image shown in Figure 5.

**Shum Wan Landslide**

The Shum Wan landslide (Hong Kong) occurred in 1995 triggered by an exceptional rainfall (381 mm/30 h and 846 mm/13 days) and favoured by the discharge of water (350 to 470 L/s) collected along a road. The landslide removed about 26,000 m³ of weathered ash and tuff, 12,000 of which did not leave completely the failure surface. A thick clay seam, composed prevalently of kaolinite, controlled the failure. We performed both 2D and 3D modelling for this event.

**Shum Wan 2D Model**

This simulation has been performed on the basis of the information available from GEO Report No. 178 (GEO 1996). We included here the presence of a groundwater table. Hydraulic conditions are considered for the initial stability and this is followed by the spreading stage during which the initial pore pressure can change accordingly to the permeability and the internal deformation of the material during the motion. Hydraulic conductivity during motion is assumed isotropic and equal to 0.1 m/s. The adopted mesh is formed by linear triangular elements with side length of about 1 m.

We adopted a Mohr Coulomb constitutive model with parameters defined for each material (based on data from table in Figure 14 of the GEO Report) and constant elastic parameters ($E =$ Young’s modulus $= 10^8$ Pa, $\nu =$ Poisson ratio $= 0.23$).

A reduction factor term (equal to 0.5) has been applied only to the plasticity parameters for the displacement of the mass at the contact with the flat deposition area at the extreme left hand boundary of the profile (A-A’ in Figure 8).

The failure surface (Figure 10) is a result of the simulation but the result is clearly controlled by the initial stratigraphy (GEO 1996) that we prepared considering the presence of the clay layer (Figures 8 and 9). After equilibrium is reached under elastic conditions, we allowed for
the plasticization to occur and the computation proceeds by the runout algorithm. As a consequence, the upper part of the slope starts moving pushing and eroding the material at the front.

<table>
<thead>
<tr>
<th>Material</th>
<th>Bulk density kN/m³</th>
<th>Friction angle °</th>
<th>Cohesion kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Landslide material</td>
<td>19.0</td>
<td>25.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2. Failure surface</td>
<td>12.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>3. Topographic surface</td>
<td>16.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Figure 5: Ortho-photos of the Frank rock avalanche with the extreme limit (white line) of the material as from the numerical simulation (taken from material made available for the Benchmarking Exercise). The table summarizes the properties adopted for the numerical simulation.

Figure 6: Material thickness (in m) computed at successive time steps, until complete stop, for the simulation of the Frank rock avalanche (positive values: deposition; negative values: source area). Initial discretization of the source volume includes 1501 elements.
Figure 6 (Con’t): Material thickness (in m) computed at successive time steps, until complete stop, for the simulation of the Frank rock avalanche (positive values: deposition; negative values: source area). Initial discretization of the source volume includes 1501 elements.

The lower part of the material moves as a thin layer and the upper material progressively override it (Figure 10). About half of the material stops along the slope.

The final deposit is thicker than observed (Figure 9) along the simulated profile but this is the result of material dispersion in the transversal direction that cannot be modelled in our fully 2D simulation.
Figure 7: Velocity vectors (m/s) computed at successive time steps for the fully 3D simulation of the Frank rock avalanche.
Figure 8: Cross section of the Shum Wan landslide showing the 6 lithological subdivisions adopted for the 2D numerical modelling. The values for the properties are summarized in the table. The material is defined by 4241 elements.

<table>
<thead>
<tr>
<th>Material</th>
<th>Bulk density kN/m³</th>
<th>Friction angle °</th>
<th>Cohesion kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Fill Nam Long Shan Road</td>
<td>20.1</td>
<td>40.0</td>
<td>10.0</td>
</tr>
<tr>
<td>2 Tuff above clay</td>
<td>19.0</td>
<td>26.0</td>
<td>2.0</td>
</tr>
<tr>
<td>3 Superficial Tuff</td>
<td>19.0</td>
<td>26.0</td>
<td>2.0</td>
</tr>
<tr>
<td>4 Tuff with clay</td>
<td>19.0</td>
<td>22.0</td>
<td>0.0</td>
</tr>
<tr>
<td>5 Tuff below failure surface</td>
<td>19.0</td>
<td>26.0</td>
<td>8.0</td>
</tr>
<tr>
<td>6 Hard tuff</td>
<td>19.0</td>
<td>38.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Figure 9: Pre-failure slope profile and comparison between observed and computed final profiles as obtained from the fully 2D numerical simulation of the Shum Wan landslide.
Figure 10: Results of the 2D modelling for the Shum Wan landslide. Cross section and the lithological subdivisions are the same as in Figure 8. Results are presented at different time steps.

**Shum Wan 3D Model**
A fully 3D model has been performed on this landslide by using 5 m long and large, and 1 m thick cells. The topography is the one obtained by the dataset distributed for the Benchmarking Exercise. The adopted lithological subdivision is simplified with respect to that adopted for the 2D analyses. The material in the source area, described by 2997 elements, is considered homogeneous and the spreading surface is differentiated from the initial failure surface from which the material is released. The properties are those reported in Table 1 with constant elastic parameters ($E$ = Young’s modulus = $10^8$ Pa, $\nu$ = Poisson ratio = 0.23).

**Table 1: Material properties adopted for the 3D modelling of the Shum Wan landslide**

<table>
<thead>
<tr>
<th>Material</th>
<th>Bulk density (kN/m$^3$)</th>
<th>Friction angle ($^\circ$)</th>
<th>Cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Landslide material</td>
<td>19.0</td>
<td>36.0</td>
<td>3.0</td>
</tr>
<tr>
<td>2. Failure surface</td>
<td>20.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>3. Topographic surface</td>
<td>22.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
We present the results in terms of velocity vectors, the thickness of the material during spreading and the geometry of the moving mass at different time steps during the failure. Incomplete validation and calibration of the model has been performed because of the absence of a deposition limit for the event.

Figure 11: Results of 3D model for the Shum Wan landslide. The landslide mass at different time steps is shown in dark blue. Observed deposit limits are in lower right inset GEO (1996).

The moving mass moves at the maximum velocity of about 12 to 12.8 m/s when the material reaches the slope toe. When the material reaches the flat area at the slope toe, it starts accumulating and velocity vectors show a divergent pattern on the left hand side of the landslide. The maximum runout distance is observed along the maximum slope direction. The final deposit reaches a maximum thickness of about 10 m in a very specific point at the slope toe, but in this area, it ranges, on average, between 5.7 and 7 m. Along the lower half of the slope, the numerical model computes up to 3.2 m of deposition.
Figure 12: 3D view of the 3D finite element mesh with the position and depth of the material for the Shum Wan landslide. Results for the fully 3D simulation are reported at different time steps. The 3D results can be compared to the 2D plan views shown in Figure 9.

**Thurwieser Rock Avalanche**

The Thurwieser rock avalanche (Central Italian Alps) involved about 2 Mm$^3$ of dolostone and black limestone that detached from about 3,600 m a.s.l. without any relevant meteorological
The September 18th, 2004 event was recorded on videos, photos and by seismic stations. A video and some photos were taken from a group of alpinists (Rozman et al. 2004) from the peak of Mt. Cevedale. Another video was taken from the Forcellino ridge, some 5.5 km to the southwest.

Through these documentations, we reconstructed the initial phases of the detachment of the rock mass from the peak (about 2 minutes) and we obtained the duration of the runout event (ca. 1'25") and the average and instantaneous velocities along different sectors at different points of the runout path. The seismic record lasts about 150 sec.

A small volume of material was deposited along the almost flat glacier surface and most of the debris was deposited along the narrow valley between 2400 and 2450 m a.s.l. with an average 25 m depth. For a total duration between 75 and 90 seconds, we obtain a mean velocity of 38 m/s, with maximum reaching 65 m/s. The computed fahrboschung is 21°.

We performed a 3D simulation by using a discretization mesh with 4840 cells of 20 m in length and width, and 3 m in thickness. The topography is described by a 20 m cell DEM.

We adopted a subdivision of the surface in different materials to simulate the landslide spreading stage because of the presence of the glacier in the upper and intermediate part of the path and of rocky outcrops and glacial debris in the intermediate and lower path. The landslide material is considered homogeneous and the spreading surface is differentiated from the initial failure surface from which the material is released. The properties are those reported in Table 2 with constant elastic parameters ($E =$ Young’s modulus $= 10^8$ Pa, $\nu =$ Poisson ratio $= 0.23$).

Table 2: Material properties adopted for the 3D modelling of the Thurwieser rock avalanche

<table>
<thead>
<tr>
<th>Material</th>
<th>Bulk density (kN/m$^3$)</th>
<th>Friction angle ($^\circ$)</th>
<th>Cohesion (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Landslide material</td>
<td>19.6</td>
<td>40.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2. Failure surface</td>
<td>26.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>3. Topographic surface ice</td>
<td>22.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>4. Topographic surface debris</td>
<td>3.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

In the following Figures (Figures 13, 14 and 15), we present some of the results in terms of: maximum computed velocities compared with those computed from the available video records (Figure 13), velocity vectors, and thickness of the material during spreading and the geometry of the moving mass at different time steps during the failure.

The validation of the model is performed by comparing the observed final thickness of the deposit with the model results (see Figure 15). Furthermore, the total time duration according to our model is comparable with the duration estimated from a sequence of photos and two movies of the rock avalanche event.

Eventually, the available estimates of the maximum front velocity have been used for a more complete dynamic calibration (see Figures 13 and 14).
Figure 13: Computed maximum front velocities for the 3D simulation of the Thurwieser rock avalanche. Computed velocities are compared with values derived from the available videos. Error bars show the uncertainty associated with the video determinations because of the possible separation of the dust and debris fronts.

Figure 14: Velocity vectors computed in the 3D simulation of the Thurwieser rock avalanche along the glacier and at the glacier cirque limit. Vectors show a clear horizontal direction in correspondence of the change in slope. This caused a partial airborne trajectory of the debris as shown also in video records at this path position.
Figure 15: Comparison of final deposit as from fully 3D numerical modelling and from field measurements and observations

**Arvel Rock Fall-Avalanche**

Landslide plowing can be relevant at the flow front, especially where sharp changes in slope occur (e.g. passing from steep slopes to flat plans). Landslide erosion can occur both at the front and along the lateral margin, and can often be suggested by the presence of a splash zone around the deposit (Heim 1932; Cruden & Hungr 1986; Voight & Sousa 1994; Abele 1997; Crosta et al. 2004; McDougall & Hungr 2005) and are sometimes at the origin of distal flows.

We decided to use the Arvel rockfall-rock avalanche (Choffat 1929; Jaboyedoff 2003) as a case study to verify and validate model capabilities especially for what concerns erosion and deposition of material.

Enough reliable data exist for this case study, which are relevant to the pre- and post-failure topography, the surficial and deep deformation of alluvial deposits involved by the landslide. The landslide occurred on March 14, 1922 in the Canton Vaud (Switzerland) along a 120 m high rocky cliff of a limestone quarry. The valley bottom was covered by alluvial deposits, mainly composed of almost completely saturated gravelly silt, sand and peat.

On March 14, after some precursors, a major rockfall detached from the cliff and a large talus developed at the slope toe. The rock avalanche volume amounts to about 614,000 m$^3$ and a 300 m long sector of the alluvial deposits was folded and faulted by the pushing action of the rock avalanche at the front of the more coarse debris. The maximum horizontal distance from the top of the main scarp to the tip of the deposit (Figure 16) is about 337 m, for a total maximum drop height of 240 m.

The fahrboschung is 35.5°, and taking into account for the disturbed alluvial deposits, it becomes 19.5°.

We ran three sets of 2D simulations assuming an initial value of about 30 m for the maximum
thickness of the alluvial deposits. Two series of numerical simulations have been run by assuming a completely dry alluvial deposit or an almost saturated deposit.

![Figure 16: Longitudinal profile along the central axis of the Arvel rockfall/rock avalanche modified after Choffat (1929). Pre- and post-failure geometries are shown together with deformations induced within the alluvial deposits along different profiles.](image)

The adopted mesh for the model construction was formed by 38,000 triangular finite elements with an average size of 2 metres.

We show the results for three different models which assume: 1) dry materials (both for the landslide and alluvial deposits) with higher cohesion; 2) saturated alluvial deposits and strength parameters reduced during runout to half the initial value; 3) same as 2) but with no strength reduction. Figure 17 shows. For model 3, the material moving along the slope, hitting the alluvial deposits and the successive sinking in and plowing of the valley infilling.

Comparing the three final geometries (Figure 18) of the landslide mass with the actual observed geometry, we note that in case of dry materials the landslide mass remains compact with a surface characterized by a regular slope. The maximum erosion depth is the lowest among the values from the different models.

For saturated conditions, lowering the strength of moving material causes the deposit to be stout but the upper surface is concave showing a good agreement with field observations in terms of erosion, maximum debris distance and longer alluvial deposit remobilization.

For saturated conditions and constant strength, the mass results more elongated (e.g. shortest tail distance and highest elevation), the general upper surface is concave but the farthest half of it is almost sub-horizontal and aligned with the surface of the ploughed alluvial deposits.
Figure 17: Movement of the debris mass as obtained at different time steps from the 2D simulation of the Arvel event, considering saturated alluvial deposits and no strength reduction (model 3).

Figure 18: Final deposit geometries as from the fully 2D numerical simulations of the Arvel rock fall – avalanche performed for 1) completely dry materials; 2) and 3) quasi saturated alluvial deposits with and without strength reduction, respectively. In grey the deposit geometry as observed by Choffat (1929). The thick black lines show the computed final geometry; dashed lines show the pre-failure profile of the alluvial deposits. Results are similar to the previous model even if the general geometry is different.

The maximum velocity is always observed at the toe of the cliff and along the initial part of the alluvial plain. Maximum velocities range from 25.7 m/s (model 2) to 25.3 m/s (model 1) and 24.3 m/s (model 1). We always observe a rapid freezing of the moving mass and a final pattern of the velocities showing the formation of a sub-circular front instability.

CONCLUSIONS
Long runout landslides are among the most critical when hazard zonation is involved. Assessment of spreading geometry and processes is extremely complex both because of the topographic constraints and of other controlling factors (e.g. water content, lithology).

We tried to consider some of these factors in a series of simulations performed through a fully 3D finite element code. We demonstrate that some of the results agree with those from experimental tests (granular dam break) and observed case studies for which a reliable set of observations are available. We chose different types of phenomena ranging from small scale physical tests to large debris and rock avalanches which developed both as thin large deposits and as thicker deposits.

Modelling of the granular step collapse are considered important because of the detailed available data and the possibility to study both the evolution in time and space also at very
low stress levels. Results of these simulations and of the other case studies demonstrate that the model is able to perform robustly in all these conditions and can be of major help both in the assessment of future events and in the understanding of the different processes.

The physical and mechanical meaning of the adopted model seems also to help in the calibration phase because values of the controlling variables are reasonably similar to real values as defined through physical characteristics of the materials. From the simulations of the Arvel case study which include the effect of material entrainment we observe that, in case of the presence of saturated sediments, the erosive process is more intense, the runout is longer and the deformation of alluvial deposits moves farther to the front of the rock-fall–avalanche debris. The excess pore pressures generated within the alluvial deposits move rapidly in the front of the moving landslide mass. Concluding, we demonstrated numerically how entrained or plowed material can control the evolution of rock and debris avalanches both in terms of geometry of deposit and velocity, and these change more when sediments are dry or saturated. A limitation to the applicability or a drawback of the developed modelling approach, with respect to the shallow water approach, consists in the computational time that is still relevant. Nevertheless, the possibility to use this approach for modelling the interaction between moving material and removable or deformable obstacles is to be considered a major advance for hazard zonation and for the design of passive countermeasures, as well as for the possibility to simulate erosion and deposition or the role of water saturated materials.

REFERENCES


ACKNOWLEDGEMENTS
The authors are indebted to Paolo Frattini for collecting GPS data for the deposit of the Thurwieser rock avalanche and to Michel Jaboyedoff for introducing them to the Arvel case study. T. Budovic is thanked for providing the photos of the Thurwieser peak failure. The Technical Committee for the Benchmarking Exercise and the corresponding authors are acknowledged for making available data for the Frank and the Shum Wan landslides.
BENCHMARKING TITAN2D MASS FLOW MODEL AGAINST A SAND FLOW EXPERIMENT AND THE 1903 FRANK SLIDE

Scott Galas, Keith Dalbey, Dinesh Kumar and Abani Patra
Department of Mechanical and Aerospace Engineering
University at Buffalo, New York, USA

Michael Sheridan
Department of Geology
University at Buffalo, New York, USA

Abstract: This paper presents benchmarking results for computer simulations of two scenarios with greatly different length scales: the 1903 Frank Slide (km scale) and a deflected sand flow experiment (m scale). The computer program used was TITAN2D, which was developed for the purpose of simulating dry granular avalanches over digital elevation models of natural terrain. TITAN2D combines best-in-practice numerical methodologies for simulations of such flows, with digital elevation models of natural terrain supported through a Geographical Information System (GIS) interface. The TITAN2D tool provides "snapshots" of each flow's progression in terms of the depth of material and its (x,y) velocities at each computational cell for discrete time intervals. It also yields several flow statistics, including area covered, maximum flow height, and average velocity, throughout time. The TITAN2D code also simulated a sand flow deflected by a change in slope. In both the Frank Slide and the sand flowing down an incline plane, the runout time of the simulated and actual flows were similar but the simulated flow spread about 20% more than the actual flow. This study shows that the TITAN2D tool can provide very high quality simulations using modest computational resources and that the uncertainty in outcome can be quantified and minimized.

BACKGROUND

Numerical models of granular flows termed “shallow-water” are based on the Saint-Venant (1871) equations for shallow-water flow. These equations have been successfully applied to dry and wet granular flows such as debris avalanches, block-and-ash flows and snow avalanches (Savage & Hutter 1989; Iverson 1997; Gray et al. 1999; Denlinger & Iverson 2001; Iverson & Denlinger 2001; Mageney-Castlenau et al. 2002; Pitman et al. 2003a; Denlinger & Iverson 2004; Iverson et al. 2004; Patra et al. 2005). Many of the models for simulating debris and rock avalanches based on the Saint-Venant equations use a Coulomb-type friction term at the interface between the material and the topography and an intergranular friction term that remains constant for the duration of the simulation. An excellent summary of this type of granular flow is presented by Pudasaini & Hutter (2007).

Unfortunately, the choice of an appropriate friction coefficient (or angle) presents a challenge to modelers due to the heterogeneity of the mobilized material. The uncertainty in this critical value is generally very large. Fortunately, the input data and actual outcomes used in this paper were provided as part of a benchmarking exercise conducted in conjunction with the 2007 International Forum on Landslide Disaster Management in Hong Kong. Therefore, the data and boundary conditions for the two examples reported here are very well constrained providing a good framework for model evaluation.
TITAN2D MODEL

TITAN2D is a computer program developed by the Geophysical Mass Flow Group at the University at Buffalo (www.gmfg.buffalo.edu) for the purpose of simulating dry granular avalanches over digital elevation models of natural terrain (Pitman et al. 2003b; Patra et al. 2006). The program is designed for simulating geological mass flows such as landslides, debris avalanches (Sheridan et al. 2005), and some types of volcanic pyroclastic flows (Rupp et al. 2006). TITAN2D combines numerical simulations of a flow with digital elevation data of natural terrain supported through a Geographical Information System (GIS) interface.

The TITAN2D program is based upon a depth-averaged model for an incompressible Coulomb continuum, a “shallow-water” granular flow. The conservation equations for mass and momentum are solved with a Coulomb-type friction term for the interactions between the grains of the media and between the granular material and the basal surface. The conservation equations solved by TITAN are given as:

\[
\frac{\partial}{\partial t} \left( \tilde{U} \right) + \frac{\partial}{\partial x} \left( \tilde{F}(\tilde{U}) \right) + \frac{\partial}{\partial y} \left( \tilde{G}(\tilde{U}) \right) = \tilde{S}(\tilde{U})
\]  

where:

\[
\tilde{U} = \begin{bmatrix} h \\ hV_x \\ hV_y \end{bmatrix}
\]

is the vector of conserved state variables (with \( h \) = flow depth, \( hV_x \) = x-momentum, \( hV_y \) = y-momentum).

\[
\tilde{F}(\tilde{U}) = \begin{bmatrix} hV_x \\ hV_x^2 + \frac{1}{2} k_{ap} g_z h^2 \\ hV_x V_y \end{bmatrix}
\]

is the mass and momentum fluxes in the x-direction (with \( hV_x \) = mass flux in the x-direction, \( hV_x^2 + \frac{1}{2} k_{ap} g_z h^2 \) = x-momentum flux in x-direction, and \( hV_x V_y \) = y-momentum flux in the x-direction).

\[
\tilde{G}(\tilde{U}) = \begin{bmatrix} hV_y \\ hV_x V_y \\ hV_y^2 + \frac{1}{2} k_{ap} g_z h^2 \end{bmatrix}
\]
is the mass and momentum fluxes in the y-direction (with \( hV_y = \) mass flux in the y-direction, \( hV_xV_y = \) x-momentum flux in the y-direction, and \( hV_y^2 + \frac{1}{2}k_{ap}g_zh^2 = \) y-momentum flux in the y-direction).

\[
\bar{S}(\bar{U}) = \begin{bmatrix}
g_xh - hk_{ap}\text{sign}(\partial V_x/\partial y)\left(\frac{\partial}{\partial y}(g_zh)\sin\phi_{int} - \frac{V_y}{\sqrt{V_x^2 + V_y^2}}\max\left(g_z + \frac{V_y^2}{r_y},0\right)h\tan\phi_{bed}\right)
g_yh - hk_{ap}\text{sign}(\partial V_y/\partial x)\left(\frac{\partial}{\partial x}(g_zh)\sin\phi_{int} - \frac{V_x}{\sqrt{V_x^2 + V_y^2}}\max\left(g_z + \frac{V_x^2}{r_x},0\right)h\tan\phi_{bed}\right)
\end{bmatrix}
\]  

is the vector of driving and dissipative source terms (with \( g_zh = \) driving gravitational force in the x-direction, \(-hk_{ap}\text{sign}(\partial V_x/\partial y)\partial/y (g_zh)\sin\phi_{int} = \) dissipative internal frictional force in the x-direction, and \(-\frac{V_x}{\sqrt{V_x^2 + V_y^2}}\max\left(g_z + \frac{V_x^2}{r_x},0\right)h\tan\phi_{bed} = \) dissipative basal frictional force in the x-direction; similar terms for y-direction); where: \( k_{ap} = \) active/passive lateral stress coefficient term, and “active” \( k_{ap} \) assumes a smaller value within a diverging flow, while “passive” \( k_{ap} \) takes on a larger value and means that the flow is converging.

The resulting hyperbolic system of equations is solved using a parallel, adaptive mesh, Godunov scheme (Patra et al. 2005). The Message Passing Interface (MPI) Application Programmers Interface (API) allows for computing on multiple processors, increasing computational power, decreasing computing time, and allowing for the use of large data sets (Patra et al. 2006). Adaptive gridding allows for the concentration of computing power on regions of special interest. Mesh refinement captures the complex flow features at the leading edge of the flow, as well as locations where the topography changes rapidly. Mesh unrefinement is applied where solution values are relatively constant or small to further improve computational efficiency.

The model used in TITAN2D assumes a pile of granular material, pulled down the slope by gravity. Friction between particles and between particles and ground resist this momentum. Governing equations for this model, the conservation of mass and conservation of momentum, are solved using numerical solution methods, e.g. finite volumes, etc. The direct outputs of TITAN2D are flow depth and momentum. These may then be used to compute, at different points, field observable variables like run-up height, inundation area and flow duration (Sheridan et al. 2005; Rupp et al. 2006).

TITAN2D operates via a python-scripted Graphical User Interface (GUI). Through this interface, the user inputs the parameters needed to successfully run the program such as pile dimensions, starting coordinates, internal and bed friction angles, and simulation time. The simulation is computed on a Digital Elevation Model (DEM) of the desired region and results
can be displayed through the TITAN2D viewer utilities, or other visualization software packages. The TITAN2D viewer utilities are designed to present end-users with a clear representation of various properties of the mass flow such as pile height and velocity magnitude. The attributes embedded in the data elements that constitute the polygonal mesh are color-coded and applied as a texture over the terrain.

FRANK SLIDE SIMULATION

Construction and Properties of Source Material Pile
TITAN2D initiates a simulation from one or more piles of granular materials using a size, shape and location chosen by the operator. We built the initial, or source, material pile for the Frank Slide scenario using the standard method of TITAN2D - namely, using Elliptic Paraboloid shaped piles. These material piles have horizontal (xy-plane) elliptical cross-sections, with parabolic vertical cross-sections. By appropriately overlaying several piles (with varying sizes, positions, and orientations) a good approximation of the actual source material for this flow results (see Figure 1). When several piles are overlapped in TITAN2D, the height of material at a particular point within that region is taken as the largest of the component pile heights at that spot, rather than summing them together. Keeping this in mind, we arrived at a best approximation of the initial material pile by guessing maximum heights for each component pile, then iteratively adjusting them, until the height at each point best fit that of the actual source pile. A natural consequence of this approach is that the overall volume and centroid location are well preserved.

Table 1: Properties of actual vs. reconstructed source material piles

<table>
<thead>
<tr>
<th></th>
<th>TITAN2D Reconstructed Pile</th>
<th>Actual Source Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Centroid location (x,y)</td>
<td>(718.0250, 1464.9)</td>
<td>(699.5807, 1452.4)</td>
</tr>
<tr>
<td>Volume (m$^3$)</td>
<td>3.6001e+007</td>
<td>3.6001e+007</td>
</tr>
<tr>
<td>Area covered (m$^2$)</td>
<td>404400</td>
<td>694800</td>
</tr>
</tbody>
</table>

A critical modification made to our source pile was to scale its base dimensions (major and minor radii) by 80%. This moved the west edge of the pile away from the ridge crest line. Without this modification, a substantial portion of material in the simulated flows moves in the wrong direction (to the west) over the ridge. This could be caused by several factors. While shrinking the size of the pile's base, we were careful to maintain its volume and centroid location as much as possible. Table 1 details a specific comparison between our “reconstructed" pile and the real material pile, provided in the DEM.
Figure 1: Comparison of actual and TITAN2D-reconstructed source material piles used in the Frank Slide scenario

**TITAN2D Frank Slide Simulation Results**
The information of this section pertains to the “baseline” run for this flow scenario. The input parameters of this run were selected by a trial-and-error process in which several TITAN2D runs were executed with various inputs, and the resulting flow inundation areas compared to that of the provided flow outline. This “baseline” case thus represents the closest match of the predicted realizations to the actual areas effected by this flow event. In a later section, we present a more formal optimization procedure that more precisely identifies optimal values of two parameters (starting location and bed friction) that give the “best” match of flow coverage areas.

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Maximum Height (m)</th>
<th>Centroid (x,y) Location (m)</th>
<th>Major Radius (m)</th>
<th>Minor Radius (m)</th>
<th>Orientation Angle, major axis relative to x-axis (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>101.0</td>
<td>(688.5, 1630.7)</td>
<td>296</td>
<td>160</td>
<td>65</td>
</tr>
<tr>
<td>2</td>
<td>223.7</td>
<td>(628.5, 1542.7)</td>
<td>273.6</td>
<td>182.4</td>
<td>140</td>
</tr>
<tr>
<td>3</td>
<td>35.7</td>
<td>(896.5, 1518.7)</td>
<td>380</td>
<td>140</td>
<td>125</td>
</tr>
<tr>
<td>4</td>
<td>151.5</td>
<td>(808.5, 1458.7)</td>
<td>212</td>
<td>156</td>
<td>150</td>
</tr>
<tr>
<td>5</td>
<td>101.5</td>
<td>(900.5, 1306.7)</td>
<td>288</td>
<td>176</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>202.5</td>
<td>(652.5, 1342.7)</td>
<td>244</td>
<td>136</td>
<td>110</td>
</tr>
<tr>
<td>7</td>
<td>106.1</td>
<td>(660.5, 1570.7)</td>
<td>152</td>
<td>144</td>
<td>15</td>
</tr>
</tbody>
</table>

Inputs to the Frank Slide simulation include: 1) bed friction angle of 14 degrees, 2) internal friction angle of 40 degrees, and 3) starting pile data. To configure the complex shape of this pile we specified: the number of piles, maximum pile heights, pile centroid locations, major and minor pile radii, pile orientation, all of which are listed in Table 2.
Figure 2: Average flow velocity (m/s) over time. This plot shows the essential flow to be complete after approximately 55 seconds.

Figure 3: Area covered (km$^2$) during Frank Slide simulation

Figure 4: Maximum flow height (m) during Frank Slide simulation
Importantly, the volume-averaged speed of the flow settled to an insignificant value after 55 seconds, indicating that the majority of the material involved in this flow had slowed to a near stop (Figure 2). Therefore we ran our baseline simulation for 60 seconds. On realization, time is longer than one minute, but with identical initial conditions as the baseline. The maximum flow thickness (Figure 4) and area covered (Figure 3) showed little change after 30 to 40 seconds.

In plan view, Figure 5 shows the placement and height distribution of the initial material pile and a record of the maximum flow depth over the course of the entire simulation, both superimposed onto the provided Frank Slide flow outline. The perimeter of the starting pile is slightly inside of the provided flow outline and the final distribution of the flow has spread more than the actual flow boundary displayed in the figure.

(a) Initial pile position and height (m) at time $t=00m00s$
(b) Record of maximum material height (m) throughout time

Figure 5: Initial material pile and record of maximum flow depth superimposed over flow outline for simulation of Frank Slide using TITAN2D.

**Computational Resources/Run-time Information**

This TITAN2D run was performed on a single processor, Compaq Presario V2000 laptop computer, with OS: Linux/Ubuntu 7.04 (see Table 3 for additional runtime information).

<table>
<thead>
<tr>
<th>Command being timed</th>
<th>&quot;./titan&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>User time (seconds)</td>
<td>1440.47</td>
</tr>
<tr>
<td>System time (seconds)</td>
<td>4.50</td>
</tr>
<tr>
<td>Percent of CPU used by this job</td>
<td>96%</td>
</tr>
<tr>
<td>Elapsed (wall clock) time (h:mm:ss or m:ss)</td>
<td>24:51.58</td>
</tr>
</tbody>
</table>

Table 3: Runtime information for TITAN2D Frank Slide "baseline" simulation

Time slices of the simulated flow properties show detailed changes in the flow characteristics along its path. Figure 6 illustrates data obtained after 30 seconds of flow time. Figures 6(a) and 6(b) show that the simulated pile has moved into the valley and the maximum thickness is in the center of the flowing mass. In the velocity vector plots of Figures 6(c) and 6(d), it can be seen that the x-velocity is concentrated along the southern edge of the flow and the
largest y-velocity is concentrated at the trail of the flow on its northeastern edge. Using this type of visual output, it is possible to monitor detailed movement in the flow including the back flow at later stages as the flow moves down to the east. This type of information is useful for hazard prediction regarding potential damage from avalanches of this type.

Figure 6: TITAN2D realization of Frank Slide at time: t=30s. (a) Material height superimposed over actual flow outline, (b) contour plot using logarithmic scales to accentuate flow features near the boundaries, (c) down-flow velocity (Vx), and (d) cross flow velocity (Vy). Note that the bottoms of the velocity scales represent negative values for the vectors.

**Sensitivity Study: Optimizing Material Coverage within Given Flow Outline**

With results from our baseline case, explained above, we then sought to determine the effect that varying the parameters bed friction angle and initial pile location has on the primary measurable outcome of this exercise, namely the degree to which the simulated and actual avalanche-effected areas coincide. We attempt to characterize this effect by performing a sensitivity study in which these parameters are varied in conjunction with one another for each of 8 additional TITAN2D simulations (in addition to our baseline case already run). The results of which (namely the similarity between predicted and actual flow boundaries) are
compared in such a way that an optimal choice of bed friction and pile position parameters can be made that results in the best matching flow coverage areas.

The measure that we use to determine the degree of “matching” of TITAN-simulated and the real flow inundation areas can be defined explicitly as: the Intersection of the two flow areas divided by their Union. See Figure 7 for method illustration.

![Figure 7: Graphical illustration of the method used to determine the correlation of simulated and actual flow coverage areas. Assuming RED = simulated flow and BLUE = actual flow, then the PURPLE = Intersection of the two, and the degree to which these flow areas coincide or “match” is taken as their Intersection divided by their Union (total colored area).](image)

**Table 4:** Combinations of parameters used in our sensitivity study TITAN2D runs showing correlations of simulated flow-coverage areas to actual flow outline

<table>
<thead>
<tr>
<th>Run Number</th>
<th>Bed friction angle (degrees)</th>
<th>Displacement along ridge (m)</th>
<th>Simulation/flow outline correlation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>14</td>
<td>0.0</td>
<td>74.5%</td>
</tr>
<tr>
<td>1</td>
<td>10</td>
<td>-179.4</td>
<td>47.8%</td>
</tr>
<tr>
<td>2</td>
<td>18</td>
<td>-179.4</td>
<td>50.0%</td>
</tr>
<tr>
<td>3</td>
<td>18</td>
<td>179.4</td>
<td>58.4%</td>
</tr>
<tr>
<td>4</td>
<td>10</td>
<td>179.4</td>
<td>64.6%</td>
</tr>
<tr>
<td>5</td>
<td>14</td>
<td>-131.3</td>
<td>62.2%</td>
</tr>
<tr>
<td>6</td>
<td>16.928</td>
<td>0.0</td>
<td>68.0%</td>
</tr>
<tr>
<td>7</td>
<td>14</td>
<td>131.3</td>
<td>71.8%</td>
</tr>
<tr>
<td>8</td>
<td>11.072</td>
<td>0.0</td>
<td>71.0%</td>
</tr>
</tbody>
</table>
Our baseline case used a bed friction angle of 14 degrees and an initial pile centroid position of (718.025 m, 1464.90 m). We chose to vary the bed friction angle within a range of 10 to 18 degrees. We chose to vary the pile centroid position in a direction that approximately parallels the local ridge, which we specifically define as that direction given by the unit vector: \( \hat{d} = -0.2494 \hat{x} + 0.968 \hat{y} \). The magnitude of pile displacement varies within a range of -179.39 m (in a predominantly southern direction) to +179.39 m (in the opposite, northern direction). This displacement range was chosen such that it is equivalent to the average radius of the initial material pile that we used in our baseline case. The range in bed friction angle was arbitrarily chosen. Figure 8 is for a schematic representation of the parameter sample space.

![Parameter Sample Space](image)

Figure 8: Parameter sample space showing combinations of bed friction angle and pile position along ridge that were used to determine an optimal combination that best matches flow coverage area when compared to the actual flow outline. The combinations of parameters, here labeled as points 0 to 8, were chosen to represent a broad coverage of the sample space.

The optimal combination of "bed friction angle" and "initial pile location" (that give the best matching of the simulated with the real flow coverage areas) was determined by finding the maximum value of a Response Surface fitted over the corresponding parameter space (Figure 9). This surface (described by the function \( f(x, y) = a_0 + a_1x + a_2y + a_3xy + a_4x^2 + a_5y^2 \)) shows the correlation between real and simulated flow coverage areas for various combinations of bed friction and pile placement parameters. The polynomial coefficients of \( f(x, y) \) were determined from a least-square fit of the surface function to the known correlation values produced by a small set of TITAN2D runs, as shown in Table 1 (which comprise our sensitivity study). With the coefficients evaluated, the proper form of the surface function is \( f(x, y) = -0.0634 +0.1172x +0.0008y -0.0000xy -0.0043x^2 -0.0000y^2 \).
The highest point on the surface (giving the maximum correlation value) occurs at bed friction angle of 13.46 degrees and pile centroid displacement (from baseline position) of +51.66 m, and has a value of 75.79%. Thus, the optimal parameters differ somewhat from our baseline case, and the “best” flow coverage area that TITAN2D can produce (when compared to the provided flow outline) clearly remains at less than an 80% match (when considering the effect of the combined variation of bed friction angle and pile placement).

Figure 9: Response surface constructed over the parameter space (bed friction angle vs. initial pile location) illustrates the degree of “matching” between real and simulated flow coverage areas. The surface is defined by the function $f(x,y) = a_0 + a_1x + a_2y + a_3xy + a_4x^2 + a_5y^2$, where the coefficients $(a_0, a_1, ..., a_5)$ were determined from a least-squares approximation using the correlation values obtained from a set of nine TITAN2D runs (the baseline, plus eight additional realizations). The combination of these parameters giving the best correlation between flow coverage areas is shown. The maximum correlation achieved is 75.79%, occurring at a bed friction angle of 13.46 degrees and a pile centroid displacement from the baseline position of +51.66 m.

**Optimization Results: Record of Material Distribution**

Figure 10 shows the results for a TITAN2D simulation run using the optimized pile position and bed friction angle as determined from the above sensitivity study. The optimized bed friction angle was determined to be 13.4640 degrees; and the best pile initial position was determined to be at a displacement (from “baseline” position) of +51.6646 (m) in the direction parallel to that of the local ridge, defined above as: $d = -0.2494 \hat{x} + 0.9684 \hat{y}$. The intersection/union correlation method (as shown in Figure 7) produced a value of 75.79% (larger than any value in Table 4) for the simulation in which we chose the optimal values of bed friction angle and pile position.
It should be noted that the bed friction of natural avalanches depends not only on the flow's mass but also upon the nature of the surface material over which they move. TITAN2D is capable of utilizing a GIS to incorporate spatially variable bed friction values from map data (Stinton et al. 2006).

Figure 10: Optimized placement of initial material pile and resulting record of maximum flow depth through time superimposed over flow outline.

**Verification of Solution Independence of Grid-Resolution**

Figure 11 shows screenshots of the TITAN2D adaptive computational grid as it appears at two discrete times during the Frank Slide “baseline” TITAN2D run. The grid is self-adaptive, concentrating computational power where it is needed, usually around flow edges or locations where mass or momentum flux becomes large. Areas with little change in momentum or near constant amounts of material do not require such a fine mesh.

Figure 11: TITAN2D's adaptive computational grid showing the change in grid refinement with time. Grid cells are refined only where necessary.

It is important to verify that the simulation results in this test do not depend on further refinement of the computational grid. To this end we performed an additional TITAN run using a grid resolution that is approximately twice that of the runs illustrated in this paper in order to ascertain the effect of a higher resolution grid on our simulation outcomes. The metric we choose for comparison was the outlines of the flows. The flow outline produced at
a low resolution shows very little difference in the predicted flow coverage area with that of a flow run at a high grid resolution (Figure 12). We conclude that this study used a sufficiently refined computational grid.

Figure 12: Flow outlines produced at low and high grid resolutions. This comparison shows that doubling the original grid resolution has very little effect on the overall predicted path of the flow.

DEFLECTED SAND FLOW SIMULATION

Simulation Setup

DEM Regeneration
The DEM for this simulation required reconstruction because the DEM that the benchmarking exercise supplied did not provide a large enough location for TITAN2D to initiate the flow. A larger area was needed to allow sufficient area as the starting position of the sand pile. To accomplish this we used MATLAB to generate a new DEM that represents the full-sized experimental apparatus with all of the dimensions obtained from the provided drawings. The resolution of our reconstructed DEM is $dx = dy = 0.00288927$ m. This scale is equivalent to the x resolution of the provided DEM (noting that the $\hat{y}$ resolution of the provided DEM is slightly finer, at $dy = 0.0028878$ m). We smoothed the DEM by setting the elevation at each point equal to the average of its adjacent neighbors. Explicitly, we calculated $z_{x_i,y_i} = 0.25(z_{x_{i+1},y_i} + z_{x_{i-1},y_i} + z_{x_i,y_{i+1}} + z_{x_i,y_{i-1}})$ for every $(x_i,y_i)$, and looped 20 times. This smoothing only affected the region of the DEM where the planes intersect; sharp edges could cause problems with the TITAN2D computation. Extremely steep topographic gradients that could potentially cause errors in the computations are generally removed by smoothing.
**TITAN2D Input Parameters**
All of the input values used for this simulation were specified in the provided drawings. These include: pile geometry, initial pile placement and orientation, bed friction angle of 32 degrees, and internal friction angle of 34 degrees.

**Simulating the Release Box**
To conduct this simulation with TITAN2D we created a virtual representation of the side walls of the release box using the following logic. The material height in each wall cell was forced to be equivalent to that of the immediately adjacent normal cell. For this case, “normal” means a non-wall cell. We set the velocity at the wall (in a direction parallel to it) equal to zero (i.e. \( V_{x\text{wall}} = 0 \)) and the velocity perpendicular to the wall equal to the negative of the velocity in the adjacent normal cell (i.e. \( V_{y\text{wall}} = -V_{y\text{norm}} \)). We also added the source term \([0.5\tan(\phi_{\text{bed}})gzh]\) to the x-momentum equation to approximate the friction that the material experiences when rubbing up against the wall.

**Simulation Results**
Our deflected sand flow simulation was run for 3.5 seconds. As with the Frank Slide simulation, the volume averaged flow speed reached a maximum and then decreases regularly reaching a very low value by the end of the run time (Figure 13). In the case of the sand flow, the maximum speed of 0.96 m/s was reached at about 0.6 seconds. The change in slope of the velocity plot at about 1.3 seconds corresponds to the time step when most of the sand had reached the trough of the device and was deflected down the crease in the apparatus.

![Figure 13: Volume averaged flow velocity (m/s) for the deflected sand flow simulation](image)

To assess the suitability of TITAN2D to represent the sand flow experiment, we made a series of images showing a side-by-side comparisons of the experimental contour plots and the TITAN2D simulation results for each 0.14 second interval in time. Figure 14 shows three of the time slices in the mid range of the simulation when the flowing sand reached the crease and begins to be deflected to the left. In all of the paired images of flow thickness contours, the TITAN2D simulation and the experimental flow followed a similar pattern. However the simulated flow spread laterally somewhat more than the experimental flow.
Figure 14: Inclined plane deflected sand flow sequence. Experiment contour plots compared with TITAN2D results, TITAN2D plots have contours at 0.005m, 0.01m, 0.02m, ..., 0.10m. Results for t=0.42s, t=0.56s and t=0.84s are shown.
Variations of simulation results through time showing the area covered by the flowing sand and the maximum thickness of the sand flow are analogous to the outcomes from the Frank Slide simulations, only scaled down. Plots of the area covered by the flowing material and the flow thickness reach almost constant values near the end of the simulation. On realization, for times of being longer than 1 second, the flowing area stays between 1.9 and 2.0 m² (Figure 15). The maximum flow height decreased rapidly to a minimum value of less than 0.06 m at 0.75 seconds, bulked to a value of about 0.075 m at about 1.5 seconds, then slowly decreased to about 0.06 m at the end of the simulation (Figure 16). As a conclusion, we assume that the sand flow simulation was essentially complete in the 3.5 seconds of our computer run.

Figure 15: Area covered by flow (m²)

Figure 16: Maximum flow height (m)

**Computational Grid: Deflected Sand Flow Scenario**

Figure 17 illustrates the Deflected Sand Flow scenario. The screenshot was taken at some arbitrary point during the flow. Unlike the adaptive grid that was used in the Frank Slide scenario, this uniform grid does not change over time. The mesh size of this grid is: dx=dy=0.0166 m.
SUMMARY
As previously stated, TITAN2D solves the conservation equations for mass and momentum using a Coulomb-type friction term to represent the interactions between particles of a medium and between the moving mass and the basal surface. The standard approach in TITAN2D is to represent the source of a flow as either an initially static pile, or in the form of a flux source, that is, material that actively extrudes from the ground. Complex pile shapes can be simulated, as we have done in the case of the Frank Slide, by overlaying several piles onto one another. Once defined, the material is subject to a driving gravitational force, with its motion being resisted by particle-particle and particle-ground interactions (represented by internal and bed friction angles, respectively). The direct outputs of TITAN2D are mass and momentum within each grid cell at each time step, from which, field observable variables such as flow run-up height, inundation area, and duration can be calculated.

Frank Slide
The complex geometry of the initial mass of material for this flow scenario was simulated using several overlapping elliptical paraboloid piles, which began the simulation in a static state. In this case, the flow essentially came to rest after approximately 55 seconds. The coincidence of the simulated and actual flow outlines was calculated (using the approach described above) to be approximately 75%. Even in the “optimized” simulation, the predicted flow path deviated slightly from reality by spreading laterally outside the flow’s actual boundaries. We showed that doubling the grid resolution had little effect on the position and shape of the outer boundary of the flow.

Deflected Sand Flow
In this case, our approach was somewhat non-standard for TITAN2D, that is, we simulated the trapezoidal shaped pile (noting again that TITAN2D normally defines piles as elliptical paraboloids). Another departure from the normal method of operation of TITAN2D, was the construction of the confining side-walls of the experimental release box. The effect of the walls was simulated directly in TITAN2D’s code (via modifications to the governing
equations for those particular cells that were designated “wall” cells). This effect was not modeled by altering the DEM. All other simulation conditions were obtained directly from the provided information. The simulated flow essentially stopped at 3.5 seconds, with a maximum height of 0.06 m at this time. We note that, at its conclusion, the simulated center of mass is displaced somewhat from that shown in the experimental results. The simulated flow, in this case, as it did in the Frank Slide, also shows more lateral spreading than is present in the actual case.

REFERENCES


ACKNOWLEDGEMENTS

This work was supported by National Science Foundation research grants ITR-(ASE+EVS) ACI-0121254 and EAR-0439093.
TWO MODELS FOR ANALYSIS OF LANDSLIDE MOTION: APPLICATION TO THE 2007 HONG KONG BENCHMARKING EXERCISES

Oldrich Hungr, Mika McKinnon and Scott McDougall
Department of Earth and Ocean Sciences
University of British Columbia, Vancouver, Canada

Abstract: The University of British Columbia (UBC) group attempted most of the benchmarking exercises. The main conclusions from this experience are summarized at the conclusion of this paper. They show that shallow problems using frictional material can be simulated quite reliably. Both the 2D model DAN and the 3D model DAN3D provide comparable results, if used with the same input parameters (although a user-specified path width function is needed for the former. The models overestimate runout and underestimate deposit thickness in problems dominated by spreading and those with thick source volumes. Entrainment of material from the path and the ability to change rheology depending on the path material can be important. Flow velocity is often not available in case histories, but is important for the selection of the optimal flow resistance model.

INTRODUCTION
The UBC group carried out analyses of most of the benchmarking exercises using two models: DAN (Hungr 1995) and DAN3D (McDougall 2006). We met with somewhat mixed success in reproducing the behaviour of the various examples. This paper summarizes some of the lessons we learned in doing the exercises.

Description of the Models
The pseudo-three-dimensional model, DAN, is described in Hungr (1995). However, since that original publication, the model was upgraded by including the active/passive earth pressure equations of Savage & Hutter (1989), by adding the ability to entrain material from the path (for which the 1995 governing equations already made an allowance) and by providing a choice of normal and vertical reference columns. The 3D model, DAN3D, is completely described by McDougall (2006) and, in a simpler form by McDougall & Hungr (2004; 2005).

The specific features of the two models are listed as follows:

DAN (Hungr 1995)

- An integrated one-dimensional Lagrangian solution.
- Linear interpolation to maintain continuity (no numerical damping).
- Ability to vary the width of the flow by a user-prescribed path width function, while maintaining volume continuity (pseudo-3D).
- Open rheological kernel, allowing the use of frictional (with constant pore-water pressure ratio), plastic, Bingham, Voellmy and other rheologies.
- Possibility to change material rheology along the path.
- Possibility to entrain material from the flow path, according to a user-prescribed erosion depth. The entrainment rate is proportional to flow depth and velocity and is set so that
fully specified entrainment depth occurs once a given point on the path is over-run by the entire current volume of the landslide.

- Option to choose vertical or normal reference columns.
- Possibility to choose between Rankine, Savage – Hutter and modified Savage – Hutter earth pressure equation (under development, see Hungr 2007 in review).
- Possibility to send the slide mass into a ballistic trajectory at steps in the path (under development).

The model has been in use for more than 10 years and has been applied probably to more than 100 case histories. It sometimes suffers from numerical instability, which is found to have little influence on the results in terms of the motion of the main mass – as long as it is not too strong.

DAN3D (McDougall 2006)

- An integrated two-dimensional Lagrangian solution.
- Use of meshless Smooth Particle Hydraulics (SPH) to maintain continuity (no numerical damping).
- Accurate numerical approximation of arbitrary source volume geometry.
- Use of arbitrary, moderately-smoothed digital terrain model of the path.
- Open rheological kernel, allowing the use of frictional (with constant pore-water pressure ratio) and Voellmy rheologies.
- Possibility to change material rheology along the path (presently controlled by elevation).
- Possibility to entrain material from the flow path, according to a user-prescribed erosion depth and erosion rate. The entrainment rate is proportional to flow depth and velocity and is set by the user by trial and error to ensure that fully specified entrainment depth occurs where required.
- Use of the Savage – Hutter earth pressure equation.
- “Steering” of the flowing mass is based on the assumption that the principal stress axes are parallel with the local direction of motion.

The following two flow resistance relationships were used by both models in this study:

1. Frictional resistance: Here, the resisting shear stress at the base of the flow equals:

   \[ \tau_{cs} = -\sigma_z (1 - r_u) \tan \phi \]  

   where \( \phi \) is the dynamic basal friction angle and \( \sigma_z \) is the total normal stress. The pore water pressure ratio, \( r_u \) is assumed constant and equals \( u/\sigma_z \), where \( u \) is the basal pore water pressure.

   If \( r_u \) can be assumed constant, the basal stress relationship remains frictional (i.e. the total normal stress and shear stress remain proportional). Equation [1] can then be simplified to include only one independent variable, a bulk basal friction angle, \( \phi_b \), where \( \tan \phi_b = (1 - r_u) \tan \phi \):

   \[ \tau_{cs} = -\sigma_z \tan \phi_b \]  

   [2]
The Voellmy resistance model combines frictional and turbulent behaviour:

\[ \tau_{\text{sh}} = -\left( \sigma_{f} f + \frac{\rho g v_{x}^{2}}{\bar{\zeta}} \right) \]  

where \( f \) is the friction coefficient and \( \bar{\zeta} \) is the so-called turbulence parameter (equal to the square of the Chézy coefficient). The first term on the right side accounts for any frictional component of resistance and has the same form as Equation [2] (\( f \) is analogous to \( \tan \phi_{b} \)). The second term accounts for all possible sources of velocity-dependent resistance.

Selected aspects of the application of the two models to the Benchmarking Exercises are described in the following sections.

**Dam Break Equation**

When DAN or DAN3D is applied to an example similar to that specified, using the given basal shear strength and zero internal strength, the result is closely comparable to the Mangeney et al. (2000) solution (Figure 1). Recent work by our group showed that movement of spreading landslides with internal strength, such as the dam-break scenario in sand, is strongly influenced by the assumed earth pressure model (Hungr 2007, in review). Common assumptions regarding the distribution of longitudinal normal stress in integrated flow models can lead to serious errors, as shown in Figure 2. Figure 2 shows the comparison of a small-scale laboratory experiment in sand with DAN analyses, using four different earth pressure assumptions: hydrostatic (no internal strength), Rankine (principal stresses parallel to bed), Savage-Hutter (principal stress rotated) and Savage-Hutter modified for the influence of steep depth gradient, typical of the dam-break flow (Hungr 2007, in review). The Savage-Hutter assumption tends to strongly overestimate spreading and runout in this problem configuration.

The above-mentioned phenomenon is likely to play a role during the first second of the dam outbreak, while depth gradients are very large. However, at longer times (10, 20 seconds), the flow becomes shallow and the difference between the Savage-Hutter and modified Savage-Hutter results becomes negligible. The modified algorithm has not yet been implemented in DAN3D.

![Figure 1: Dam-break benchmarking exercise, flow profiles at t=10 to 30 seconds, compared with the closed-form solution (from McDougall 2006). Note: different friction angle than in the assigned exercise).](image)
Figure 2: Back-analysis of a small-scale laboratory dam-break experiment in sand, using four alternative earth pressure assumptions. Both the basal and internal friction angles were measured as 30.9º by tilting tests. The dots show the experimental results (from Hungr 2007, in review).

Deflected Flow Experiment
We reproduced the geometry of the experiment as well as possible, to match the contour plan provided by the Lausanne group. We then used a basal friction angle of 32º and an internal friction angle of 34º and a total volume of 30,000 cm³. The resulting distribution of deposits at time $t=3.3$ seconds is shown in Figure 3 and compared with the last frame of the sequence provided by the Lausanne group.

Our result is accurate in terms of width of the deposit, but our deposit is about 10% longer. Also, the maximum thickness of the deposit is 8 cm, compared with 10 cm in the experiment. The centre of gravity of the deposit appears to correspond quite well. The time sequence is comparable, but appears to be out of sync by about 0.1 seconds. Our flow reaches the intersection of the two planes at about 0.6 seconds after opening of the gate and the basal platform at 1.3 seconds. A second analysis using DAN with assumed uniform path width produced similar depth distribution and timing of travel (Figure 4).

Figure 3: Comparison of calculated (red) and measured deposit distribution. Contour interval 1 cm, 0.5 cm maximum.
Fei Tsui Road Landslide

We used two different frictional rheologies, separated at an elevation of 40 m: a bulk friction angle of 20° on the slide scar (this corresponds to the static slope stability analyses during failure) and a $\phi_b$ of 35° on the street pavement below. In both cases, the internal strength of the material was 35°. The rationale is that pore-water pressure was acting on the detachment bedding plane before the sliding block dilated, but the movement over the paved runout surface was essentially a dry frictional flow. The slide volume was 14000 m$^3$, which is assumed to include bulking. The result of the DAN3D analysis is shown in Figure 5.

The analysis was repeated using DAN, assuming that the flow path widens from 40 m at the source to 100 m in the deposit. In order to account for the irregular inclination of the path, DAN had to be used with vertical reference columns, instead of the standard normal columns.
The timing of the slide and the distribution of deposits is similar as in the 3D solution (Figure 6).

Both solutions indicate much greater longitudinal (and - in case of 3D - lateral) spreading of the slide mass. Since spreading is an important component of motion of this landslide, the Savage-Hutter earth pressure equation, built into both of our algorithms, has an inherent tendency to overestimate spreading (Hungr 2007, in review). We have so far not been able to complete the analysis using the prototype modified Savage-Hutter algorithm in DAN, due to instability problems.

The Frank Slide of 1903
The Voellmy model produced the best results. Figure 7 shows a parametric study, exploring the influence of the two Voellmy parameters on the debris runout. The best results in terms of overall runout distance and debris distribution is obtained using an \( f \) of 0.1 and a \( \xi \) of 500 m/s² (debris unit weight of 20 kN/m³). The back-calculated friction parameters give some indication of the average pore-water pressure conditions during the landslide motion. Assuming that the dynamic effective friction angle of rapidly shearing rock fragments is about 30º, the friction coefficient of 0.1 used in the Voellmy solution implies an average pore-water pressure ratio of 0.83. The mean bulk friction angle of 17º corresponds to an \( n_u \) of 0.47.

High pore water pressure ratios are certainly possible in those parts of the path where the landslide over-rides saturated soils of glacial, colluvial or alluvial origin. They are not likely to exist on the upper slopes and, especially within the source area of the rock slide. As the failing rock mass fragments, it expands in volume. The large new void space generated by fragmentation must be initially dry. To account for this, we attempted some analyses where the source area was modeled using a dry frictional sliding model, transferring to the Voellmy model on the path below. It was found that, in order to obtain the requisite runout distance with this model, the mean friction angle in the source area would have to be as low as 15º. With such a provision, the model produces results similar to the uniform Voellmy analysis mentioned above.
The low friction angle is justified, considering the conclusions of Cruden & Krahn (1978) who found that highly polished pre-sheared bedding planes in the Turtle limestone could have friction angles as low as this value. Rapid shearing of a discontinuity plane under normal stresses corresponding to more than 100 m column of rock would, presumably, lead to intensive polishing of the surface, especially if pre-sheared by previous flexural slip.

Unfortunately, information regarding movement velocity is not available. However, based on eyewitness interviews, McConnell & Brock (1904) conclude that the “time that elapsed between the first crash and complete rest did not exceed 100 seconds, and may have been somewhat less”. The Voellmy model shown in the centre of Figure 7 gives an overall
movement duration of less than 100 seconds. The corresponding distribution of maximum flow velocity is given in Figure 8.

A uniform frictional analysis was also attempted. The constant “bulk friction angle, $\phi_b$ of 17$^\circ$ had to be used, to obtain the requisite runout distance. The frictional model is not as satisfactory as the Voellmy result for two reasons: 1) The travel time is considerably shorter (about 40 seconds). 2) The frictional results show the typical forward-tapering deposit distribution, predicting considerable thickness of deposits in the proximal region, near the Crowsnest River. The Voellmy model predicts greater thickness in the distal region, near the highway and this is closer to reality.

Figure 9 shows a two-dimensional analysis of the slide using DAN (Hungr 1995). The path width was based on the mapped extent of the damaged area as shown in Figure 7. In general, the 2D results correspond very well with the 3D results, both in terms of the deposit distribution and slide velocity and duration. The results reported here have been derived recently. However, the main conclusions are the same as those reported by Hungr & Evans (1996).

Figure 8: Maximum velocity contours, constant Voellmy rheology, $f=0.1$, $\xi=500$ m/s$^2$

Figure 9: Frank Slide, Voellmy analysis, $f=0.1$, $\xi=500$ m/s2. Profiles plotted at 10 second intervals. The dashed line is the input width function.
**Tsing Shan Debris Flow, 1990**

We carried out a number of trial runs using DAN, specifying a small initial slide of about 500 m$^3$ and an average 1 m entrainment depth down to an elevation of 120 m, yielding a final volume of approximately 20,000 m$^3$. An example of a frictional run, using a bulk friction angle of $\phi_b=18^\circ$ is shown in Figure 10.

A similar debris distribution, but with a maximum velocity of only 20 m/s was obtained using a constant Voellmy rheology with an $f = 0.15$ and $\xi = 500$ m/s$^2$.

Figure 11 shows a comparison of calculated and measured distribution of deposits along the slope, expressed as lag rate (negative yield rate) in m$^3$/m path length. The measured values have been compiled by Hungr et al. (2005), from reports by King (1996).

A three-dimensional result for the Voellmy case, using DAN3D appears in Figure 12.

![Figure 10: Tsing Shan debris flow 1990. Frictional analysis, $\phi_b=18^\circ$. Depth profiles at 10 sec intervals, depth exaggerated 10x.](image)
Figure 11: Tsing-Shan 1990 Debris Flow, Lag rate comparisons. (a) Field data (Hungr et al. 2005, data from King 1996); (b) Frictional, $\phi_b = 18^\circ$ (different colours indicate different earth pressure equations); (c) Voellmy, $f = 0.15$, $\xi = 500$.

The travel path of the lower deposit is incorrect, possibly due to excessive smoothing of the topography, or because the present topography has been modified by the 1990 event. In all analyses, the deposits are too closely concentrated. The reason for this is probably reworking of the deposits by water in the later stages of the debris flow, which has not been considered in the analysis.
Shum Wan Road Landslide
After trying a number of alternatives, we found two best-fitting models: frictional with a $\varphi_b$ of 22° and Voellmy with an $f = 0.2$ and $\zeta = 200$ m/s$^2$. Both produce similar deposits, although the Voellmy model has velocities about 30% less than the frictional. Both models over-predict lateral spreading towards the left margin. This may result from greater internal stiffness of the sliding mass that assumed in the 3D model. Two-dimensional DAN models made previously by Hungr et al (1999) produced similar results.

Figure 12: DAN3D analysis using the Voellmy parameters, $f = 0.2$, $\zeta = 500$ m/s$^2$

Thurwieser Rock Avalanche
We first tried some uniform Voellmy parameters that worked well for rock avalanches in the past (see, for example, our solution of the Frank Slide, which worked well with $f=0.1$ and $\zeta=500$ m/s$^2$). Figure 14 shows three examples.

Figure 13: Shum Wan Road landslide, Frictional, $\varphi_b = 22°$. The deposit depth contours at 2 m intervals.
All of these runs are obviously unsuccessful. Either the material is too mobile and runs outside the map, or too sluggish and accumulates at the base of the upper slope. Our previous experience with flow of rock avalanches over glacial ice indicated that the resisting stresses are well simulated with a low $f$ (typically 0.05) and high $\xi$ (typically over 1000 m/s$^2$).

Reasonably good results have been obtained by R. Sossio, using a frictional rheology with a $\phi_b$ of 25-28° on and just below the source scar, a Voellmy rheology with an $f=0.1$, and $\xi=1000$ m/s$^2$ on the glacier ice and again frictional rheology with a $\phi_b$ of 28° on the morainal surfaces below the glacier (Figure 15). The peak velocity of 50 m/s below the glacier compares reasonably with the velocities measured in the video record (Crosta et al. 2007).

The material “runs away” to the east at the source. This is again a demonstration of internal stiffness of the sliding body in the initial stages of motion, before complete disintegration of the rock mass. The expanded slide volume is 2.22 million m$^3$. Further details are given by Crosta et al. (2007).
The analysis is unable to simulate the thin, widespread deposit on the morainal slopes below the glacier.

**Small Debris Avalanches**

The 1999 Sham Tseng, the 2000 Tsing Shan and the 2005 Tate’s Cairn debris avalanches-debris flows could be reasonably analysed using a frictional model with a $\phi_b$ of less than 20° or a Voellmy model with an $f$ of 0.2-0.3 and a $\zeta$ of 200 to 500 m/s$^2$. Very small erosion depths of much less than 0.5 m had to be used, in order not to exaggerate the slide volume. The main difference between the two rheologies is the peak velocity. The runout distance is determined primarily by the friction coefficient, $f$.

**CONCLUSIONS**

The following general conclusions can be drawn from these exercises:

1. Both models are quite reliable for shallow frictional flow, as characterized by the deflected flow experiment. This conclusion is in agreement with previous findings (e.g. McDougall 2006).
2. Both models consistently produce comparable results, given the same input parameters. This shows that neglect of lateral momentum equilibrium, inherent in DAN, is not very important.
3. The models overestimate both longitudinal and lateral spreading in problems where the energy input is dominated by this mechanism, such as the dam break problem or the Fei Tsui road landslide.
4. In many cases involving thick initial mass, the analysis shows excessive lateral spreading, particularly in the early stages of movement, when the real moving mass is probably still semi-coherent.
5. In some cases it is crucial to vary the material rheology along the path. This is shown by the Thurwieser rock avalanche, where substantially lower resistance properties must be specified for the flow over the glacier.
6. Material entrainment from the path is very important in cases like the 1990 Tsing Shan debris flow and can be modelled satisfactorily, provided that the erosion depth can be reliably estimated beforehand.
7. The frictional and Voellmy models often produce comparable results, although the latter exhibits lower velocities. Collection of velocity data from case histories is important for selection of optimal models. Other rheologies, such as Bingham, have not been tried in this study. Previous experience indicates that they are unlikely to produce superior results in cases such as those studied here.

**REFERENCES**


ACKNOWLEDGEMENTS
The Authors are grateful to the Organizers for the opportunity to participate in this exercise.
APPLICATION OF 2D-FINITE VOLUME CODE FLATMODEL TO LANDSLIDE RUNOUT BENCHMARKING EXERCISES

Marcel Hürlimann
Department of Geotechnical Engineering and Geosciences

Vicente Medina and Allen Bateman
Sediment Transport Research Group
Technical University of Catalonia, Barcelona, Spain

Abstract: The present paper summarizes the main results obtained from the Landslide Runout Analysis Benchmarking Exercise organized during the International Forum on Landslide Disaster Management. We applied the 2D-finite volume code FLATModel to different cases including two validation cases and several real debris flows that recently occurred in Hong Kong. The results show that FLATModel represents an accurate and user-friendly tool to correctly simulate such complex phenomena.

INTRODUCTION
Detailed simulation of landslide dynamics, in particular debris-flow behaviour, is a difficult task and modelling of both over-steepened bed slopes and on smooth fans needs special requirements. Many different one-dimensional (1D) or two-dimensional (2D) models were presented and especially 2D simulations strongly improved insights in the dynamic behaviour of such flows (Laigle & Coussot 1997; Denlinger & Iverson 2001, 2004; Chen & Lee, 2003; Pitman et al. 2003; Denlinger & Iverson, 2004; McDougall & Hungr 2004; Zanuttigh & Lamberti 2004; Pudasaini et al. 2005; Rickenmann et al. 2006).

The present study applied our 2D-finite volume code FLATModel, which has been developed in the past years by the fruitful collaboration of the hydraulic and geotechnical departments indicating the need of multidisciplinary approaches to resolve such a complex process.

DESCRIPTION OF FLATMODEL
In the following, the general characteristics of FLATModel will be summarized. More comprehensive explanations on the governing equations, computational scheme, terrain considerations etc. can be obtained in other recent publications on this numerical code (Bateman et al. 2007; Medina et al. 2008).

The general classification of FLATModel can be summarised as a monophasic, two-dimensional model with the z-axis normal to the bed. The development of the 2D governing equations starts from three-dimensional conservation laws applied to monophasic material and follows the typical depth integration process (Vreugdenhil 1994). In FLATModel, after applying integration in the direction normal to the bed and corrections related to slope and curvature, we obtain an equations system similar to the one proposed by Iverson & Denlinger (2001), Rickenmann et al. (2006), Pitman et al. (2003) or McDougall and Hungr (2004). However, the model of Iverson & Denlinger (2001) includes lateral stresses and water pore pressure that are neglected here. Finally, basic equation of
FLATModel can be expressed by:

\[
\begin{align*}
\frac{\partial}{\partial t} \begin{pmatrix} h \\ h u \end{pmatrix} + \frac{\partial}{\partial x} \begin{pmatrix} h u \\ h u^2 + g_p \frac{h^2}{2} \end{pmatrix} + \frac{\partial}{\partial y} \begin{pmatrix} h v \\ h u v \\ h v^2 + g_p \frac{h^2}{2} \end{pmatrix} &= \begin{pmatrix} 0 \\ h(g_p \tan \alpha_x - S_f) \\ h(g_p \tan \alpha_y - S_f) \end{pmatrix} \\
\end{align*}
\]

[1]

where \( t \) is the temporal coordinate; \( h \) is flow depth; \( u, v \) are the velocities in the \( x,y \) directions, respectively; \( g_p \) is the corrected gravity; \( S_f \) is the energy gradient and \( \alpha \) is the terrain surface angle. The subscripts \( x,y \) indicate vector component.

The pressure term is affected by flow streamlines characteristics, slope and curvature. Both, slope and curvature are measured in the velocity flow path direction in order to modify the two momentum conservation equations, in which the terms appear.

The normal stresses acting at the bottom or contact surface are affected by the centripetal acceleration that was included into the FLATModel by making two modifications. Firstly, adding a centripetal acceleration in the momentum pressure term to obtain the corrected gravity, \( g_p \):

\[
\begin{align*}
g_p &= g \cos \theta + \frac{|V|^2}{r_c} \\
\end{align*}
\]

[2]

where \( g \) is the acceleration due to gravity; \( \theta \) is the angle defined by the gravity and pressure gradient, or in other words, the angle defined by the horizontal plane and the velocity direction, which is tangent to the streamline; \( |V| \) is the velocity modulus and \( r_c \) is the radial curvature evaluated in the velocity direction. Then, the pressure force per unit length at the face, \( F_p \), can be computed by:

\[
F_p = g_p \frac{\rho h^2}{2} \\
\]

[3]

where \( \rho \) is the flow density.

The second modification refers to the bed normal stress, \( \sigma_b \), which is computed by:

\[
\sigma_b = g_p \rho h \\
\]

[4]

Because FLATModel calculates the depth in vertical direction, the real flow depth is determined by the correction angle, \( \theta \), in every model cell before applying the conservation equations. The same procedure also corrects the space increments \( dx \) and \( dy \) to reflect the real flow path length and to compute the real flux gradients.

**ENTRAINMENT MECHANISMS**

As an initial remark, we must state that the following technique represents a preliminary
approach to better understand entrainment mechanisms of debris flow and may provides
general patterns on this complex topic.

Our so-called “static approach” can be easily described with soil mechanics concepts. It
considers a static equilibrium between the flow frictional forces (bed shear forces) and the
basal resistance forces, $\tau_{res}$. This equilibrium should be achieved at each computational time
step:

$$\tau_b = \tau_{res}$$  \[5\]

The bed shear stress represents flow friction, while the resistant shear stress can be
represented by the Mohr-Coulomb failure criterion for an infinite slope

$$\tau_{res} = c + h \rho g \cos \theta \tan \phi_{bed}$$  \[6\]

in which $c$ is the cohesion and $\phi_{bed}$ is the bulk friction angle of the bed material. Streamlines
are assumed to be parallel to the bed in the numerical approximation, that’s why $\alpha = \theta$. Because FLATModel uses a homogeneous flowing mass, it cannot distinguish the different phases (water and sediment). Two-phase models, however, should reduce the flow depth in Eq. [6] to compute the effective stresses and not the total stresses.

FLATModel checks the equilibrium conditions of Eq. [5] in every time step. If there is no
equilibrium, the model calculates the magnitude of entrainment necessary to achieve
equilibrium by:

$$\tau_b + h_{ent} \rho g \sin \theta = c + \left[h + h_{ent}\right] \rho g \cos \theta \tan \phi_{bed}$$  \[7\]

where $h_{ent}$ is the entrainment or erosion depth. This expression can be transformed to obtain
an explicit value of $h_{ent}$

$$h_{ent} = \frac{\tau_b - \tau_{res}}{\rho g \left(\cos \theta \tan \phi_{bed} - \sin \theta\right)}$$  \[8\]

Finally, entrainment conditions can be established for a cohesionless torrent bed material, if:

$$\tau_b > h \rho g \cos \theta \tan \phi_{bed}$$  \[9\]

The meaning of this condition is that the bed shear forces have to be greater than the
resistance forces to provoke basal erosion. An additional condition regarding bed stability is
necessary to guarantee solution. This conditions is given by:

$$\cos \theta \tan \phi_{bed} > \sin \theta \Rightarrow \tan \phi_{bed} > \tan \theta$$  \[10\]

This means that the terrain collapses if its slope is greater than its internal friction angle.
Once $h_{ent}$ has been calculated, FLATModel recalculates the momentum of the cell due to the
fact of the incorporation of new mass with a low quantity of momentum. Obviously the
resultant velocity diminishes due to entrainment.
DAMBREAK SCENARIO

Methodology Applied
The dam-break scenario presented by Mangeney et al. (2000) involves movement of debris in a wide channel inclined at an angle, $\theta$. The resistance to the debris flow generated on the channel bed is characterized by an angle of friction, $\phi_{\text{bed}}$.

Here, one principal test was carried out, where the initial depth of debris ($H$) is taken as 10 m, the channel bed inclination, $\theta$, and the angle of friction, $\phi_{\text{bed}}$, are taken as $\theta = 30^\circ$ and $\phi_{\text{bed}} = 25^\circ$ respectively. Simulated flow profiles were calculated at every 10-second interval, up to 30 seconds after the dam removal. Another test case was calculated to compare results for $\theta = 0^\circ$ and $\phi_{\text{bed}} = 0^\circ$.

Results
In our first example, flow movement was simulated for an initial depth of debris of 20 m, a horizontal channel and zero friction at the bed (Figure 1(a)). The flow profile simulated by FLATModel at 10 sec indicates that numerical results are identical as the analytical ones. In our second example, the initial depth of debris was taken as 10 m, the channel inclination as 30$^\circ$ and the basal friction angle as 25$^\circ$. Figure 1(b) illustrates flow profiles at 10, 20 and 30 sec indicating some minor differences between numerical and analytical results. These small differences could even be reduced by applying a second-order spatial discretization.

Figure 1: Comparison between analytical solutions of one-dimensional dam-break scenarios and numerical results calculated by FLATModel. (a) Dam-break characterised by an initial flow depth of 20 m and material of zero bed friction angle across a horizontal surface. (b) Dam-break characterised by an initial flow depth of 10 m and material with a bed friction angle of 25$^\circ$ across a surface inclined by 30$^\circ$.

USGS FLUME TEST

Methodology Applied
Sophisticated laboratory flume experiments were performed for sand avalanches to obtain
insights of dry granular flow behaviour across irregular terrain (Iverson et al. 2004). Here, we back-calculated experiment A, which was carried out for angular sand particles with an internal friction angle of about 44º and an angle of bed friction of between 20º and 23º (see Iverson et al. 2004 for more detailed information on experiment set-up and properties). Data available on the USGS experiment was pre-processed by ARCGIS and transformed into raster file of 0.001 m cell size.

Coulomb flow resistance law was implemented into FLATModel for this back-analysis. The two different friction angles 20º and 23.47º, which depend on the material of the flume, were used for bed friction, while internal friction was not incorporated.

**Results**

The comparisons between simulation results using two different basal friction angles and data from the flume experiment are given by isopach maps of flow depth at different times (Figure 2). Model predictions match many of the dynamic characteristics of the sand avalanche and errors of front velocity, final extension and depth of material are generally of minor importance.

![Figure 2](image-url)

**Figure 2:** Comparison between the laboratory experiment of a sand avalanche (left column; Iverson et al. 2004) and numerical results calculated by FLATModel (right column). Flow depth is plotted on a topographic base with a contour interval of 1 cm.

In continuation, we present an example how our stop-and-go approach can be visualised for the same simulation run as described above.
Figure 3: Visualisation of the stop-and-go approach implemented in FLATModel at different time steps using a basal friction angle of 23.47°. Green (bright) colour indicates cells in movement and red (dark) colour shows cells with no movement.

1990 TSING SHAN DEBRIS FLOW, HONG KONG

Methodology Applied
We decided to calculate the debris flow using Voellmy fluid for the flow resistance law because colluvium material may be better represented by turbulent or “granular-type” flow behaviour than by a more viscous rheology.
We applied a two-step approach to back analyse this debris flow using FLATModel. Firstly, we approximated the best-fit rheological parameters: the internal friction angle of the flowing mass, $\phi$, and the Chezy coefficient, $C_z$, using reference data such as the maximum velocity of the debris flow (~16.5 m/s), the maximum runout distance and the extension of the final deposit. Secondly, we analysed the entrainment of the debris flow by varying the internal friction angle of the bed material, $\phi_{\text{bed}}$, and the thickness of the colluvium, which limit the maximum depth of entrainment.

**Results**

Initially, we should mention that the initial topographic map of the Landslide Study Report 2/2001 (King 2001) illustrates that the movement was not confined in the higher part of the flow trajectory and a visible channel starts at about 230 m above sea level. Thus, the present mass movement may be called a hillslope debris flow, which is not confined in a torrent. This fact in combination with the rather smooth topography obtained by the DEM (5 m grid cell) may have affected the simulation.

Simulation results show an important lateral spreading, especially in the upper part of the trajectory (Figures 4 and 5). Finally, the best-fit Voellmy fluid parameters were $\phi = 11.3^\circ$, $C_z = 8 \text{ m}^{1/2}/\text{s}$ and the friction angle of the colluvium was assumed to be $\phi_{\text{bed}} = 37^\circ$; while colluvium was taken as 3 m thick (representing the maximum erosion depth of debris flow). Maximum erosion depth is defined as 3 m due to some information and photographs found in the Landslide Study Report 2/2001 (King 2001). Using these input parameter, the initial volume of 400 m$^3$ strongly increased and the final volume was about 31,000 m$^3$.

![Figure 4](image.png)

Figure 4: Simulation results for $\phi = 11.3^\circ$, $C_z = 8 \text{ m}^{1/2}/\text{s}$, $\phi_{\text{bed}} = 37^\circ$. (a) maximum flow depth, (b) maximum velocity. Depth of colluvium is taken as 3m.
Figure 5: Simulation results for $\phi = 11.3^\circ$, $C_z = 8 \text{ m}^{1/2}/\text{s}$, $\phi_{\text{bed}} = 37^\circ$. Flow depth at different time steps. Depth of colluvium is taken as 3m.

In a second phase of the back analysis, we studied the effect of different $\phi_{\text{bed}}$-values on the total volume mobilised. The initial volume was taken as 400 m$^3$ (defined by the input-file). Results indicate that final volume is very sensitive to the value of $\phi_{\text{bed}}$ and show that the approach is really delicate and very detailed data is necessary! If a certain critical volume was eroded in the upper part of the trajectory, unrealistic high volume was calculated due to continuation of entrainment in the lower part of the slope. This error may be avoided by limiting the extension of the area of possible erosion.
Finally, the entrainment approach indicates that a $\phi_{\text{bed}}$-value of about 38º would provide a final volume of 20,000 m$^3$ (final volume observed in the field).

Figure 6: Influence of $\phi_{\text{bed}}$ on the final volume. (a) 1990 Tsing Shan Debris Flow using $\phi = 11.3^\circ$, $C_z = 8$ m$^{1/2}$/s as rheological parameters and an initial volume of 400 m$^3$. (b) Results of the 1982 Pal debris flow (Medina et al. 2008) is shown for comparison. Here, initial volume was 500 m$^3$ and final volume was 5,000 m$^3$.

Additionally, simulation runs were carried out for a colluvium thickness of 5 m, thus assuming a maximum erosion depth of 5 m. The final volume using $\phi = 11.3^\circ$, $C_z = 8$ m$^{1/2}$/s as rheological parameters and $\phi_{\text{bed}} = 37^\circ$ was about 55,000 m$^3$. In comparison, the final volume for 3 m colluvium was about 31,000 m$^3$.

2000 TSING SHAN DEBRIS FLOW, HONG KONG

Methodology Applied
This case was back-calculated without the option to incorporate entrainment. Simulations were finally carried out using 1,600 m$^3$ as constant volume. Again, Voellmy fluid rheology was such as applied and the best-fit rheological parameters were approximated using as reference data the maximum velocity of the debris flow (~14 m/s in the north-branch and ~18 m/s in the south-branch), the maximum runout distance and the extension of the final deposit.

Results
Again, an important lateral spreading is visible in the higher part of the trajectory, which was not confined at all. Finally, the best-fit Voellmy fluid parameters were $\phi = 8.53^\circ$ and $C_z = 20$ m$^{1/2}$/s.
Figure 7: Simulation results for $\phi = 8.53^\circ$ and $C_z = 20 \text{ m}^{1/2}/\text{s}$. (a) maximum flow velocity, (b) depth of final deposit.

In Figure 8, vectors of maximum velocity computed in each cell are shown. This useful output of FLATModel is of particular interest in the bifurcation area of the debris flow.

Figure 8: Velocity vectors for $\phi = 8.53^\circ$ and $C_z = 20 \text{ m}^{1/2}/\text{s}$. (a) general view, (b) zoom to the bifurcation.

CONCLUSIONS
Detailed simulation of debris-flow dynamics is a complex task and can be carried out using different types of numerical codes. Here, our 2D finite-volume numerical code FLATModel was applied. Several general closing notes can be obtained from the results of the benchmarking exercises.

FLATModel is an accurate and easy-handling numerical code for simulation of debris flows and other mass movements. The pre- and post-processing of the data can be directly carried out in a GIS, which makes FLATModel very user-friendly.

The application of Voellmy fluid model presented accurate results for the real debris flows that occurred in Hong Kong by comparing simulation outcomes with field observations.
The implementation of basal entrainment could represent rather well the effect of scouring along the flow trajectory. In spite of the uncertainties regarding material properties, the final volume coincides more or less with the data observed in the field. Simulation results indicated however that the final volume is sensitive to the selected friction angle of bed material.

The “stop-and-go” mechanism enables the visualization of the dynamic behaviour and improves the understanding of the processes interacting in the accumulation zone.

REFERENCES

**ACKNOWLEDGEMENTS**
The authors would like to thank the organizing team of the benchmarking exercises. The development of FLATModel was supported by the Spanish Research and Technology Ministry, contract BTE2002-0375, the EC-project HYDRATE, contract GOCE 037024 and the Generalitat de Catalunya, research group 2005SGR00770.
BENCHMARKING EXERCISE ON LANDSLIDE MOBILITY MODELLING – RUNOUT ANALYSES USING 3dDMM

Julian Kwan and H W Sun
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

Abstract: The Geotechnical Engineering Office (GEO) of the Government of the Hong Kong SAR is responsible for combating landslide problems and mitigation of landslide risk to the community of Hong Kong. Assessment of landslide debris mobility is one of the key components to support studies of the landslide risk and for the design of landslide hazard mitigation measures where required. With the continuous technological improvement, a 3-dimensional landslide runout simulation model has been developed by the GEO. Simulations of most of the landslide benchmarking cases in the 2007 International Forum on Landslide Disaster Management have been carried out using this model, and simulation results are found satisfactory with reasonable accuracy compared with the corresponding analytical solutions, laboratory measurements and field observations/measurements in real landslides.

INTRODUCTION
A numerical simulation package “3d Debris Mobility Model” (3dDMM) for 3-dimensional landslide runout analyses has recently been developed by the Geotechnical Engineering Office, Civil Engineering and Development Department, Government of the Hong Kong SAR. The model solves two-dimensional shallow water equations using a numerical technique called Particle-in-Cell (PIC). This model has been used to simulate most of the cases in the landslide runout benchmarking exercise in the 2007 International Forum on Landslide Disaster Management. The strategy of the PIC technique involves representing the landslide debris by a number of imaginary particles and implements numerical integration over an array of Eulerian grids laid on the topography of the landslide runout path. 3dDMM adopts the Savage-Hutter theory in the calculation of the lateral earth pressure within the debris mass. Voellmy rheology and frictional rheology have been built into the model for calculation of the basal resistance to landslide motions.

MODEL DESCRIPTIONS
Equations of Motions
3dDMM is a continuum model, in which debris motions are modelled by depth-averaged momentum equations as described in McDougall & Hungr (2005):

\[ \rho h \frac{\partial u}{\partial x} = \rho h g_z + \sigma_z \left( -\frac{\partial k_{xy} h}{\partial x} \right) + \sigma_z \left( -\frac{\partial k_{xy} h}{\partial y} \right) + \tau_{xz} - \rho u E_z h \sqrt{u^2 + v^2} \]  \[ \text{[1]} \]

\[ \rho h \frac{\partial v}{\partial y} = \rho h g_y + \sigma_y \left( -\frac{\partial k_{xy} h}{\partial y} \right) + \sigma_y \left( -\frac{\partial k_{xy} h}{\partial x} \right) + \tau_{yz} - \rho v E_z h \sqrt{u^2 + v^2} \]  \[ \text{[2]} \]
The above equations refer to the orthogonal Cartesian coordinate system x-y-z, where z is in the direction normal to the ground surface. \( \rho \) is the bulk density of debris, \( h \) is the height of the debris column normal to the ground surface, \( u \) and \( v \) are the depth-averaged debris velocities in the x and y directions respectively. Similarly, \( g_x \) and \( g_y \) are the accelerations due to gravity, resolved in the x and y directions, whereas \( g \) is the gravitational acceleration in vertical direction. The last term in the momentum equations accounts for the momentum flux resulted from entrainment. Parameter \( E_r \) represents the bed-normal depth eroded per unit flow depth and unit displacement (McDougall & Hungr 2005). \( \sigma_z \) is the normal stress at the debris base:

\[
\sigma_z = \rho h (g \cos \alpha + c_z)
\]

where \( \alpha \) is the gradient of the ground to the horizontal and \( c_z \) is the centripetal acceleration due to curvature of the terrain in the direction of debris motion.

The earth pressure coefficients, \( k_x \), \( k_y \), \( k_{xy} \), and \( k_{yx} \), correspond to the tangential normal and shear stresses within the debris. They are evaluated using the earth lateral pressure coefficients obtained by the Savage-Hutter theory (Savage & Hutter 1989):

\[
k^p_{(active/passive)} = \left\{ \begin{array}{ll}
k^p_{active} & ; \varepsilon^p > 0 \\
1-\sin \phi ; \varepsilon^p = 0 \quad , \quad k^q_{passive} & ; \varepsilon^q < 0
\end{array} \right.
\]

\[
k^q_{(active/passive)} = \left\{ \begin{array}{ll}
k^q_{active} & ; \varepsilon^q > 0 \\
1-\sin \phi ; \varepsilon^q = 0 \quad , \quad k^p_{passive} & ; \varepsilon^p < 0
\end{array} \right.
\]

The Savage-Hutter theory calculates the stress coefficients, \( k^p \) and \( k^q \), in the direction of the major and minor principal stresses. The superscripts, \( p \) and \( q \), over the symbol \( k \) indicate the major and minor principal axes respectively. Parameter \( \phi \) is the bulk friction angle of the debris, and \( \delta \) is the dynamic friction angle at the debris base. The stress coefficients are evaluated according to the active and passive conditions on the principal axes. Assuming that the major principal stress aligns with the dominant deformation, 3dDMM assigns \( k^p \) and \( k^q \) based on the following criteria:

\[
k^p = \begin{cases} k^p_{active} & ; \varepsilon^p > 0 \\ 1-\sin \phi ; \varepsilon^p = 0 \\ k^p_{passive} & ; \varepsilon^p < 0 \end{cases}, \quad k^q = \begin{cases} k^q_{active} & ; \varepsilon^q > 0 \\ 1-\sin \phi ; \varepsilon^q = 0 \\ k^q_{passive} & ; \varepsilon^q < 0 \end{cases}
\]

where \( \varepsilon^p \) and \( \varepsilon^q \) are the rates of strain along the principal axes.

Having calculated the stress coefficients, \( k^p \) and \( k^q \), 3dDMM adopts the ‘standard’ Mohr circle formulations to convert the stress coefficients to \( k_x \), \( k_y \), \( k_{xy} \) and \( k_{yx} \) for each of the faces of the debris column.

The basal resistance terms, \( \tau_{x} \) and \( \tau_{xy} \) included in the momentum Eqs. [2] and [3], are calculated using the Voellmy rheology:
\[ \tau_{zx} = \frac{-u}{\sqrt{u^2 + v^2}} \left[ \sigma_z \tan \delta + \rho g \frac{u^2 + v^2}{\xi} \right] \]  \[ \tau_{zy} = \frac{-v}{\sqrt{u^2 + v^2}} \left[ \sigma_z \tan \delta + \rho g \frac{u^2 + v^2}{\xi} \right] \]

The last term in the above two equations represents the equivalent resistance acting at the base of the debris to account for turbulence effects. \( \xi \) is the Voellmy coefficient. When this term is dropped, the formulations correspond to a pure frictional rheology.

**Numerical Scheme**

3dDMM has been developed based on a numerical scheme called Particle-in-cell (PIC). PIC was first developed by the Los Alamos National Laboratory of the U.S. for computational fluid dynamics (Harlow 1963). The method was then adopted in numerical simulations of deformations of elastic-plastic materials (Sulsky et al. 1995). It has also been applied in modelling of geo-dynamic processes, such as the dynamics of sea-ice by Flato (1993), and tectonic plate movement by Moresi et al. (2002).

PIC adopts particle representation of the deformable materials concerned. Each of the particles considered in 3dDMM represents a specified volume of debris (V). Unless particles travel beyond the computational domain, the number of particles conserves in the course of the numerical calculation. Therefore, the conservation of mass is achieved, and solving of the mass conservation equation is not required explicitly in contrast to some other Eulerian numerical models. The numerical integration of the momentum Eq. [1] and [2] is implemented over an array of Eulerian grids laid on the topography. The grid dimensions are \( l \) by \( l \). Each of the particles has an influence zone of plan area equal to the grid size. The debris height and velocity in a grid are calculated by summations over the fractions of particle within the grid as shown in Figure 1.

![Figure 1: Calculation of debris velocity and height in grid](image)

3dDMM calculates debris height and momentum of individual grids, which are occupied by particles. Using this approach, finite difference equations for each of the grids are set up using a central differencing scheme. The finite difference equations are then solved explicitly, resulting in the rates of change of velocity, \( \partial u / \partial x \) and \( \partial v / \partial y \). Assuming that the rates of
change of velocity remain constant over a short period of time, \( \Delta t \), the updated grid velocities in \( x \) and \( y \) directions can be approximated by:

\[
u = u' + (\partial u / \partial x) \Delta t \quad \text{and} \quad v = v' + (\partial v / \partial y) \Delta t
\]  

where \( u' \) and \( v' \) are velocities at previous time step.

Redistribution of the updated velocities back to the particles is needed in the PIC scheme. The redistribution is carried out based on the afore-mentioned fraction-weighting method but in a reversed manner. After assigning the updated velocity to particles, the particles are advanced to new positions by means of central differencing approximation:

\[
x = x' + \Delta t (u + u')/2 \quad \text{and} \quad y = y' + \Delta t (v + v')/2
\]  

Through the above time marching procedure, the numerical calculation can be repeated to reach a specified prototype time. The numerical accuracy and stability depend on the distance of particle advancement and particles travel at large steps with respect to the grid size should be avoided. The time stepping interval should also be sufficiently short. Usually, simulations can be repeated using a smaller interval to verify the adequacy of the time stepping interval adopted. As a rule of thumb, the time stepping interval should be sufficiently small such that the particle travelling distance at each time step is limited to one-fifth of the grid size.

Another key factor to numerical accuracy and stability is the number of particles within an individual grid. If the number of particles in a grid is small, motion of particle travelling across the grid may cause large fluctuation of the instantaneous values of the local field quantities, such as debris height and momentum. However, with the fraction-weighting method adopted in the calculation of the spatial field quantities across grids, the fluctuation can be averaged out effectively. In typical simulation runs, numerical stability can be achieved where the minimum number of particles per cell is kept to be two.

3dDMM has been programmed to simulate the effects of entrainment. It calculates the erosion volume on a grid basis at every time step. The erosion volume within a particular grid \( A \), \( \Delta V_A \), is proportional to the grid velocity and debris height, \( h_A \).

\[
\Delta V_A = E_r h_A l^2 \sqrt{u^2 + v^2} \Delta t
\]  

The entrainment volume, \( \Delta V_A \), is distributed to particles using fraction-weighting method and the debris volume that a particle represents will increase accordingly.

**MODELLING RESULTS OF THE BENCHMARKING CASES**

**Dam Break Scenario**

A two-dimensional version of the 3dDMM was used to simulate the dam break scenario. Hydrostatic condition is assumed in the simulation. The cell size and particle volume are 3 m and 1 m³ respectively. Time stepping at one second interval was adopted.

The debris profiles at different time intervals after the dam removal are shown in Figure 2. The simulation results are satisfactory and generally agree with the analytical solutions using
an internal frictional angle of 25° and the same bed inclination (30°) as specified in the benchmarking exercise. However, the model does not reproduce exactly the analytical solutions at the flow front and at the top of the profile where singularity occurs. Some improvements can be obtained where smaller grid size and smaller particle volume are used.

Figure 2: The analytical solutions and the 3dDMM results

**Deflected Sand Flow**

The laboratory experiment involves releasing dry fine sands from a box placed in flume, which was set up using two inclined planes sloping at different angles. The first runout plane is inclined at 37.5° and the second one is inclined at 22°. Figure 3 below shows the geometry of the set up.

The 3dDMM simulation has been carried out using the same internal frictional angle and base friction angle as measured in the laboratory, which are 34° and 32° respectively. Other key parameters adopted in the simulation are:

(a) grid size = 5 mm by 5 mm,
(b) maximum time stepping interval = 5 x 10^-4 s, and
(c) particle volume = 5 x 10^-6 m³.

6,000 particles are used in the numerical calculation. Figure 4 shows the simulated flow patterns of the dry sand. The red dotted line represents the outline of the final sand deposition recorded in the experiment. The outline is defined by the thickness of 3 mm. In general, the calculated flow pattern matches well with the laboratory observation. However, 3dDMM calculates a slightly larger deposition area compared with the one observed in the experiment. This could be due to the assumption of lateral earth coefficient, i.e. 1 - sinϕ, for the static level ground condition as compared with the test on sloping ground.
Figure 3: The experimental set up

Figure 4: Flow patterns produced by 3dDMM

Figure 5 presents the average velocity of the dry sand flow calculated using 3dDMM. It shows the average velocity peaks at $t = 0.5$ s, at which the flow front arrives at the less-steep $22^\circ$ plane (see the flow pattern at Time = 0.4 s in Figure 4), where deceleration of the flow commences. The sand flow practically stops at 2 s after the release. At that time, the average velocity drops below 0.2 m/s.
A comparison of the measured and the calculated final locations of the centre of mass has also been made. The red cross in Figure 6 indicates the centre of mass of the final deposition calculated from the measured results, and the blue cross indicates the 3dDMM estimation. The plan distance between the two centres is 3 cm.

Figures 7 and 8 show the transverse and the longitudinal deposition profiles of the dry sand. It appears that 3dDMM over-predicts the extent of the deposition by about 0.08 m in the transverse direction. This results in a shallower predicted deposition depth at the trough of the flume. Figure 8 shows that the calculated maximum deposition depth is 0.09 m, which is 0.02 m less than the measurement.
USGS Flume Tests
The USGS reported two dry sand flow experiments using a miniature flume of 1 m long by 0.4 m wide (Iverson et al. 2004). Two experiments, Experiments A and B, were conducted with flume bed of different “topography”. Angular sands of grain sizes between 0.5 mm to 1 mm were used in Experiment A, while finer and round sands of grain sizes between 0.25 mm to 0.5 mm were used in Experiment B. Internal friction angle and basal friction angle were measured using tilt table, the reported values are summarised as follows:

<table>
<thead>
<tr>
<th>Experiment</th>
<th>Internal Friction Angle</th>
<th>Basal Friction Angle (over the irregular topography)</th>
<th>Basal Friction Angle (elsewhere)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>43.99°</td>
<td>19.85°</td>
<td>23.47°</td>
</tr>
<tr>
<td>B</td>
<td>39.39°</td>
<td>22.45°</td>
<td>25.6°</td>
</tr>
</tbody>
</table>

Figure 7: Debris deposition profiles in the transverse direction

Figure 8: Debris deposition profiles in the longitudinal direction
The sands were loaded behind a vertical wall at the upper end of the flume. The maximum thickness of the sand at the source was about 4.35 cm. The wall was equipped with a gate, and the sands were released by opening up the gate. The widths of the gate were 12 cm and 4 cm in Experiments A and B respectively.

Simulations of the dry sand flows were carried out using the reported basal and internal friction angles. The grid size is taken as 5 mm. The particle volume and time stepping interval are $2 \times 10^{-8}$ m$^3$ and 0.0001 s respectively. The model results are compared with the laboratory data in Figures 9 and 10. Contours for depths less than 3 mm are not shown in the comparisons, as the grain size of the sand particles is in the range of 0.25 mm to 1 mm. In reality, thin sheet of sands with depth < 3 mm would comprise only a few numbers of sand grains of which the dynamics is likely to be similar to that of movement of discrete particles rather than a flow of continuum as assumed in the numerical model.

Results of the numerical model generally match the overall behaviour of the sand flows observed in the experiments. The debris profiles as influenced by the three-dimensional form of the flume bed are adequately represented by the model results. Figure 9 shows that, at $t = 0.93$ s, the model predicts two streams of sand flows in localized gullies as observed in the experiment. Errors in predictions of the frontal displacement are less than 9% for all time steps. However, the model over-predicts the velocity of the flow and the extent of the deposition zone. The over-prediction of deposition extent could be a result of the simplified approach adopted in calculating the lateral pressure within the debris, which does not explicitly incorporate energy dissipation due to soil internal strain.

![Debris Depth and Model Results for USGS Experiment A](image-url)

Figure 9: USGS Experiment A
Shum Wan Landslide, Hong Kong

The Shum Wan Landslide involved a landslide mass of 26,000 m$^3$. It occurred on a 30$^\circ$ natural hillside during heavy rainfall. A planar, slab-like ground mass detached and slid down from the hillside and the landslide debris was stopped after hitting a shipyard at the slope toe. Site observations suggested that the landslide mass might remain intact during the sliding process.

A series of back-analyses was carried out using 3dDMM. The best match results are produced where frictional rheology with $\delta = 20^\circ$ was adopted. The peak average velocity of debris was 10 m/s and debris motion was found practically ceased at 25 s after initiation. Figure 11 shows the calculated deposition profile of the landslide debris.
Sham Tseng San Tsuen Debris Flow

In the morning of 23 August 1999, several landslides occurred at the natural hillside above a village called ‘Sham Tseng San Tsuen’ in Hong Kong. The landslide debris, of volume 600 m$^3$, ran into a streamcourse and developed into a channelised debris flow. The debris flow demolished several dwellings in front of the mouth of the streamcourse. Voellmy rheology was adopted in the 3dDMM simulation of the Sham Tseng San Tsuen Debris Flow, on the basis that the debris flow consisted of a high water content when travelling along a steep drainage line. The maximum gradient of some of the sections of the debris trail was over 45°, a high degree of turbulence within the debris flow is expected. Voellmy model with $\delta = 12^\circ$ and $\xi = 500$ m/s$^2$ was adopted in the back-analysis. Figure 12 shows the simulation results, in which the red dotted line indicates the extent of debris trail observed on site and the blue boxes depict the dwellings. The final deposition location estimated by 3dDMM agrees well with the site observation.

Figure 12: Simulation results of the Sham Tseng San Tsuen debris flow
Fei Tsui Road Landslide

The landslide occurred on a man-made cut along Fei Tsui Road in Hong Kong. The width and length of the landslide scar were 33 m and 90 m respectively. The average depth of the landslide mass was 15 m, and the corresponding landslide volume was 14,000 m$^3$. The landslide investigation revealed that there was an extensive kaolinite-rich layer on the base of the landslide scar dipping towards Fei Tsui Road. After the onset of the landslide, the debris ran across Fei Tsui Road and was brought to stop after hitting a reinforced concrete building (a church) on the opposite side of the road.

The basal resistance to movement of the landslide mass was calculated using frictional rheology. Within the landslide source area, a low base friction angle of 22° was used to account for the presence of the kaolinite-rich layer. In contrast, a higher base friction angle, of 35° was used for movement of the landslide debris over Fei Tsui Road. The simulated average velocity of debris is given in Figure 13. The peak average velocity is estimated to be 5.7 m/s. At $t = 8$ s, the average debris velocity drops below 0.5 m/s indicating that debris motions have virtually stopped. The debris profiles are shown in Figure 14.

![Figure 13: Average debris velocity estimated by 3dDMM](image)

Time series of debris velocity and debris depth at the southern corner of the church is presented in Figure 15. The model calculates that the landslide debris would impact on the church at a velocity of 3.8 m/s. The debris surge is estimated to be about 0.5 m high at the time of impact and debris piles up gradually against the church to a maximum depth of 3 m.

The simulated debris profile is compared with the site measurement as shown in Figure 16. The calculated displacement of debris at the landslide crown and the debris runout at the distal end compares generally well with field measurements. However, discrepancy between the simulated and the measured debris depths is observed at the location of a steeply sloping ground at the toe of the cut slope.
Figure 14: Simulated debris profiles

Figure 15: Debris depth and velocity hydrographs at the southern corner of the church
The debris flow was initiated by a landslide of about 350 m$^3$. The detached ground mass dashed down the hill for about 100 m. At Ch 100, it triggered a large landslide of about 4,000 m$^3$. The large landslide subsequently developed into a debris flow through heavy entrainment along its runout path. The debris volume was increased to 20,000 m$^3$ before reaching its final deposition. Site observation indicated that most of the debris deposited between Ch 500 and Ch 750, and hydraulic action of the surface runoff caused washout of fine-grained debris to about Ch 900.

The debris flow simulation from Ch 100 has been carried out using 3dDMM. The initial debris volume input to 3dDMM was 4,000 m$^3$. Parameters $\delta = 15^\circ$, $\xi = 500$ m/s$^2$ and $E_r = 0.0035$ m$^{-1}$ were adopted in the simulation.

Figure 17 shows the debris average velocity and the total debris volume at different time intervals after initiation. 3dDMM estimated that the debris motion basically ceased at $t = 34$ s. The simulated debris volume increases from 4,000 m$^3$ to 20,000m$^3$ during the course of the runout.

Figure 18 shows the simulated final debris profile at 40 s after the landslide. 3dDMM estimated that debris would deposit on the hillside between Ch 500 and Ch 750. Further washing out of landslide debris due to surface water flow beyond Ch 750 was not simulated.
A landslide involving failure volume of 1,000 m$^3$ occurred in August 2005 in Tate’s Cairn, Hong Kong. The landslide debris entered into an ephemeral drainage line and developed into a channelised debris flow. The runout distance of the debris flow is 330 m and the travel angle is 22°. The runout path is inclined at an angle of about 35° to the horizontal, and the inclination is gradually reduced to 12° at the debris deposition zone. At about 220 m from the landslide source, landslide debris changed its direction of movement following a sharp bend of the drainage line.

3dDMM simulation of the debris flow was carried out using Voellmy rheology with the
parameters, $\delta = 15^\circ$ and $\xi = 500 \text{ m/s}^2$. No entrainment was allowed in the simulation on the basis that the reported volume of erosion along the debris trail is merely $150 \text{ m}^3$, which is insignificant compared with the initial volume of the debris flow. Site observation revealed that debris was deposited gradually along the trail and a total volume of $200 \text{ m}^3$ of debris reached Ch 330.

The simulation provides a good match of the debris runout and the extent of the debris trial (see Figure 19). The red line in the figure depicts the debris-affected zone. However, the gradual debris deposition along the trail is not simulated, while 3dDMM estimates that the debris deposition takes place at the last $50 \text{ m}$ of the runout path (i.e. between Ch 280 and Ch 330). The maximum average debris velocity was estimated to be $11.5 \text{ m/s}$. This maximum velocity occurs where the debris reaches the sharp bend in the stream course.

Figure 19: Simulated debris profiles

**Tate’s Cairn Landslide Forward Prediction**

The forward prediction exercise concerns runout of a potential landslide involving a failure volume of $10,000 \text{ m}^3$ initiated from a landslide source area close to the source area of the 2005 Tate’s Cairn Landslide. Three sets of Voellmy parameters are given by the Review Committee for this exercise. The values of the three sets of Voellmy parameters are as follows:

<table>
<thead>
<tr>
<th>Case</th>
<th>Basal friction angle</th>
<th>Voellmy coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$8^\circ$</td>
<td>$500 \text{ m/s}^2$</td>
</tr>
<tr>
<td>B</td>
<td>$15^\circ$</td>
<td>$500 \text{ m/s}^2$</td>
</tr>
<tr>
<td>C</td>
<td>$25^\circ$</td>
<td>$500 \text{ m/s}^2$</td>
</tr>
</tbody>
</table>

Entrainment was allowed in the simulations. According to requirements of the exercise, the entrainment rate assumed in the simulations should double the volume of the debris flow. In addition, the runout analysis shall also be repeated without entrainment allowed.

**Tate’s Cairn Landslide Forward Prediction (Case A)**

Figure 20 shows the debris profiles at different time intervals after the initiation of the landslide. The purple dotted line indicates the landslide source area. The simulation was carried out with an entrainment rate, $E_r = 0.0015 \text{ m}^{-1}$. 3dDMM results indicate that the debris volume doubles at $t = 30 \text{ s}$, when debris reaches buildings at the foothill area (the clusters of...
buildings are depicted by red boxes in the figure). At $t = 40$ s, the frontal end of the debris flow reaches the boundary of DEM provided for this exercise and the debris gradually flows beyond the DEM domain afterwards.

![Simulated debris patterns](image)

**Figure 20: Simulated debris patterns**

As revealed in Figure 20, 3dDMM indicates that the potential massive debris flow would affect the road (the blue line in the figure) and the clusters of residential buildings (the red rectangles). The estimated debris velocity and depth at the affected locations are given below.

<table>
<thead>
<tr>
<th>Facility</th>
<th>Debris velocity (m/s)</th>
<th>Debris depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road at location X</td>
<td>16.7</td>
<td>3.2</td>
</tr>
<tr>
<td>Building at location Y</td>
<td>15.6</td>
<td>2.7</td>
</tr>
<tr>
<td>Building at location Z</td>
<td>7.3</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The simulation was repeated for the scenario with no entrainment. The debris flow in this simulation does not flow beyond the DEM domain, and the debris front advances at a slower pace comparing with that assuming entrainment. However, the simulation results also indicate that the debris flow would affect the road and the clusters of residential buildings at
the foothill area. Figure 21 shows the simulated flow patterns at different time intervals.

![Simulated flow patterns at different time intervals](image)

**Figure 21: The simulated flow patterns for case A (without entrainment)**

**Tate’s Cairn Landslide Forward Prediction (Case B)**

Simulations were repeated with the Case B Voellmy parameters. The final deposition patterns predicted by 3dDMM are given in Figure 22. For both entrainment and non-entrainment runs, debris stops at the location of the road (i.e. the blue line in the figure). The final debris volume in the entrainment run is 20,000 m$^3$ and the entrainment rate adopted is $E_r = 0.0018$ m$^{-1}$.

**Tate’s Cairn Landslide Forward Prediction (Case C)**

The Case C parameters correspond to a scenario with the largest basal resistance amongst the other cases. Debris was predicted to deposit on the hillside without reaching any downhill facilities. In addition, about 35% of the debris remains within the landslide source area (see Figure 22). Owing to the slow debris motion and the short runout distance, the simulation results are not sensitive to the value of $E_r$ adopted. In the entrainment run, $E_r = 0.002$ m$^{-1}$, which is 30% of the values used for the Case A simulation, was adopted, and only a minor increase of debris volume is predicted.

**2000 Tsing Shan Debris Flow**

The initial volume of the debris flow was 150 m$^3$. The debris flow was then developed into a 1,600 m$^3$ debris flow as a result of entrainment along its runout path. The debris flow bifurcated into two branches after hitting a structural steel pylon located at the head of a ridgeline.

The back-analysis of the debris flow was carried out using Voellmy rheology ($\delta = 15^\circ$ and $\xi =$...
500 m/s²). A high basal friction angle (δ =45°) was adopted at location of the pylon to simulate flow resistance due to the impact. The entrainment rate, $E_r$, was taken as 0.0011 m⁻¹ before the debris is divided into two branches, and it was taken to be 0.0002 m⁻¹ for the rest of the runout paths. Figure 23 shows the simulated flow patterns at the instances when the debris bifurcates. The simulated pattern “over-shoots” the extent of southern branch at the head of the ridgeline.

<table>
<thead>
<tr>
<th>Case B</th>
<th>Case C</th>
</tr>
</thead>
<tbody>
<tr>
<td>with Entrainment</td>
<td>with Entrainment</td>
</tr>
<tr>
<td>without Entrainment</td>
<td>without Entrainment</td>
</tr>
<tr>
<td>Time=60.0s</td>
<td>Time=50.0s</td>
</tr>
</tbody>
</table>

![Figure 22: Debris deposition depths for Case B and Case C](image)

![Figure 23: Simulated flow patterns during the course of bifurcation](image)

**Frank Slide, Canada**

Frank Slide involved a massive rock avalanche of about 36 Mm³. The rock fragments travelled down a hillside of 800 m high. The runout distance was about 3.5 km, the deposition area was 1.7 km wide with a maximum depth of 18 m.
Back-analysis shows that, when using frictional model ($\delta = 12^\circ$), 3dDMM produced results that match well with the site observations (see Figure 24). The results indicate that debris motions ceases at 55 s after the landslide initiation.

Figure 24: Calculated deposition depth of Frank Slide

**Thurwieser Rock Avalanche, Italy**

This rock avalanche happened at the Central Italian Alps, involving a total debris volume of 2 Mm$^3$. The landslide mass detached from the source area and first entered into the Zebru Glacier, then it travelled on a hummocky rock outcrop before its deposition into a valley, which is about 1,000 m below the source area.

Frictional rheology was adopted to simulate the rock avalanche. The basal friction angle within the glacier region was taken as 12$^\circ$, while the friction angle was assumed to be 27$^\circ$ for areas outside the glacier region. Calculation using 3dDMM indicates that the whole avalanche process would take about 80 s to complete (see Figure 25), and at that time, the average debris velocity could fall below 2 m/s.
Figure 25: Average debris velocity produced by 3dDMM
Figure 26 shows the simulated debris patterns, the red dotted line indicates the observed debris trail. The results show that the entire debris mass enters into the glacier region at about 20 s from the onset of the avalanche. However, 3dDMM over-estimates the extent of the debris trail at the glacier region. At $t = 40$ s, about half of the debris moved across the glacier; and at $t = 60$ s, the debris front reaches the valley where debris deposition was observed. The debris mass at the deposition zone stops at $t = 80$ s, and the maximum debris thickness at the valley at this moment is estimated to be 15 m, which is less than the observed deposition thickness (25 m).

CONCLUSIONS
3dDMM produces generally satisfactory results for the benchmarking cases reported above. The benchmarking cases comprise a wide spectrum of phenomena/characteristics of debris flows and landslides ranging from analytical solutions, small scale dry sand flows in laboratorial environments, to typical landslides and debris flows that consist of medium to high water contents, and to mega-scale rock falls such as the Frank Slide and Thurwieser Rock Avalanche. Most of the results calculated using 3dDMM agree well with analytical solutions, the site or laboratory measurements as well as findings from detailed studies of the landslide cases.

The modelling approach adopted in 3dDMM (i.e. using the PIC numerical procedure to solve the shallow water equations) is capable of simulating satisfactorily motions of landslide debris when applying frictional or Voellmy rheology for evaluating dissipation of kinetic energy of a wide range of landslide debris dynamics. This exercise also reveals some areas requiring further attention, which includes the basic assumptions adopted in the calculation of lateral earth pressure within the debris mass. Refinement such as consideration of internal strain-dependent calculations of earth pressure distribution may improve the accuracy of the simulation results.
REFERENCES
three-dimensional terrain: 2. Experimental tests.” Journal of Geophysical Research,
landslides.” Canadian Geotechnical Journal, 42, 1437-1448.
Viscoelastic/Brittle Lithosphere: Numerical Methodology and Plate Tectonic
to solid mechanics.” Computer Physics Communications, 87, 236-252.

ACKNOWLEDGEMENTS
Thanks are due to HN Wong for his continued support and guidance during the development
of the GEO model 3dDMM and in this benchmarking exercise. The authors are grateful to the
unfailing support provided by other members of the GEO team in this benchmarking exercise,
especially Thomas Wong and Florence Ko, who assisted in setting up some of the numerical
models. This paper is published with the permission of the Head of the Geotechnical
Engineering Office and the Director of Civil Engineering and Development, Government of
the Hong Kong Special Administrative Region.
BENCHMARKING EXERCISES FOR GRANULAR FLOWS

Antoine Lucas
Institut de Physique du Globe de Paris, Université Paris Diderot, France

Anne Mangeney
Institut de Physique du Globe de Paris, Université Paris Diderot, France
and Institute for Nonlinear Sciences, UCSD, San Diego, USA

François Bouchut
Département de Mathématiques et Applications, École Normale Supérieure, France

Marie-Odile Bristeau
Institut National de Recherche en Informatique et Automatique, Le Chesnay, France

Daniel Mège
Laboratoire de Planétologie et de Géodynamique, Université de Nantes, France

Abstract: For the 2007 International Forum on Landslide Disaster Management framework, our team performed several numerical simulations on both theoretical and natural cases of granular flows. The objective was to figure out the ability and the limits of our numerical model in terms of reproduction and prediction. Our benchmarking exercises show that for almost all the cases, the model we use is able to reproduce observations at the field scale. Calibrated friction angles are almost similar to that used in other models and the shape of the final deposits is in good agreement with observation. However, as it is tricky to compare the dynamics of natural cases, these exercises do not allow us to highlight the good ability to reproduce the behavior of natural landslides. Nevertheless, by comparing with analytical solution, we show that our model presents very low numerical dissipation due to the discretization and to the numerical scheme used. Finally, in terms of mitigation and prediction, the different friction angles used for each cases figure out the limits of using such model as long as constitutive equations for granular media are not known.

INTRODUCTION

Model Description
Simulation is performed using the numerical model, hereafter called Shaltop-2d, resulting from a long term collaboration between Département de Mathématiques et Applications (DMA), École Normale Supérieure (ENS, Paris) and Institut de Physique du Globe de Paris (IPGP) in France (Bouchut et al. 2003; Bouchut 2004; Bouchut & Westdickenberg 2004; Mangeney et al. 2005, 2007). This model was developed after former studies performed by our group in the context of a collaboration between IPGP, ENS-DMA and INRIA. A first model was developed based on the model developed for shallow river flows by Audusse et al. (2000) and Bristeau et al. (2001).

Mangeney et al. (2003) extended this model based on a kinetic scheme to the classical Savage-Hutter model for granular flows over sloping topography with a Coulomb-type basal friction involving either a constant friction coefficient or the empirical flow rule proposed by
Finally, the projection of the gravity field on the reference frame linked to the topography was performed during the PhD of M. Pirulli together with the active/passive earth pressure coefficient (Pirulli et al. 2007) and several basal friction laws have been tested on real events (Pirulli 2004; Pirulli & Mangeney 2007). This model after called RASH$^{3D}$ by Pirulli et al. (2007) is used in the Benchmarking Exercice by Pirulli and Scavia. The main advantage of this model is the unstructured finite element mesh best suited to deal with complex topography. However, the numerical method is only of the first order and the friction, the projection of the gravity field and the introduction of the earth pressure coefficients are not compatible with the preservation of steady state and do not respect all the properties required by the numerical resolution of the equations (see Bouchut 2004 for details about numerical methods to solve these problems). Furthermore, this kinetic model (i.e. RASH$^{3D}$ model) is based on the Savage-Hutter equations developed in a reference frame linked to the topography including only the curvature terms in the $x$ and $y$ direction. It is noted that the aforementioned problems are shared with most of the numerical models proposed in the literature.

Contrary to the classical approaches, the new model Shaltop-2d is based on the equations developed by Bouchut et al. (2003) and Bouchut & Westdickenberg (2004) developed in a fixed cartesian reference frame with the thin layer approximation (TLA) imposed in the direction perpendicular to the topography (see Figure 1). The rigourous asymptotic analysis makes it possible for the first time to account for the whole curvature tensor. Furthermore, the numerical method is of the second order requiring less refined grid to reach the same precision. The numerical method is based on the work of Bouchut (2004) and preserve the steady states as well as other requirements related to the resolution of hyperbolic equations.

The overall idea is to develop the equations in a fixed reference frame $(x, y, z)$, for example horizontal/vertical, as opposed to the equations developed by Hutter and his co-workers in a variable reference frame linked to the topography. However, the shallowness assumptions are still imposed in the local reference frame $(X, Y, Z)$ linked to the topography (Figure 1). Indeed, to satisfy the hydrostatic assumption for shallow flow over inclined topography, it is the acceleration normal to the topography that must be neglected compared to the gradient of the pressure normal to the topography. The reference frame is shown in Figure 1. The 2D horizontal coordinate vector is $x = (x, y) \in \mathbb{R}^2$ and the topography is described by the scalar function $b(x, y)$ with a 3D unit upward normal vector:

$$\vec{n} \equiv (-s, c) \in \mathbb{R}^2 \times \mathbb{R}$$

where,

$$s = \frac{\nabla_y z}{\sqrt{1 + \|\nabla_y z\|^2}}$$

and

$$c = \frac{1}{\sqrt{1 + \|\nabla_y z\|^2}}$$
where $\nabla_{xy}$ is the gradient of the topography in both directions and $\theta$ is the angle between $\mathbf{n}$ and the vertical.

In the horizontal/vertical Cartesian coordinate and for an inclined plane, formulation of the equations reduce to:

$$
\partial_t h + c \cdot \partial_x (hu) + \partial_y (hv) = 0
$$

$$
\partial_t (u) + cu \cdot \partial_x (u) + v \cdot \partial_y (u) + c \cdot \partial_x (ghc) = -g \sin \theta + \tilde{f}_x
$$

$$
\partial_t (v) + cu \cdot \partial_x (v) + v \cdot \partial_y (v) + \partial_y (ghc) = \tilde{f}_y
$$

where $h$ is the depth-thickness of the flow, $g$ is the vertical acceleration and $\mathbf{u}(u, v)$ is the depth-average velocity. The two-dimensional Saint-Venant system with topography and friction is written:

$$
\begin{align*}
\partial_t h + \partial_x (hu) + \partial_y (hu) &= 0 \\
\partial_x (hu) + \partial_x (hu^2 + gh^2/2) + \partial_y (hv) + hg \partial_y b &= hf_x \\
\partial_y (hv) + \partial_y (hv^2 + gh^2/2) + hg \partial_y b &= hf_y
\end{align*}
$$

Debris avalanche is treated here as a single-phase, dry granular flow with Coulomb-type behavior. The transition between a static state and flowing state is modeled by introducing a threshold allowing or not the material to flow. The friction forces $f(x, y, t) = (f_x, f_y)$ (see Eqs. [5] and [6]) must satisfy
The friction coefficient is either supposed to be constant (Coulomb friction law) or depending on the thickness $h$ and the Froude number of the flow $u/(gh)^{1/2}$ (hereafter called Pouliquen flow rule):

1. Coulomb friction law defined as follow:

$$\mu = \tan \delta$$

where $\delta$ is the constant friction angle.

2. Pouliquen friction empirical rule (Pouliquen 1999):

$$\mu(h) = \tan \delta_1 + (\tan \delta_2 - \tan \delta_1) \exp\left(-\beta \frac{h}{d} \sqrt{g h}\right)$$

where $\delta_1$ and $\delta_2$ are characteristic friction angles of the material, $d$ is a length scale of the order of a grain diameter, and $\beta = 0.136$ is a dimensionless parameter as it has been proposed by Pouliquen (1999). Basically, the friction parameter is increasing for small thickness $h$ and high velocity $u$. The Coulomb type friction law applied on the averaged media has been tested by comparing depth-averaged model Shaltop-2d with discrete element simulations (Mangeney et al. 2006) on the collapse of granular column of variable aspect ratio ($a = H_i/R_i$, where $H_i$ and $R_i$ are the initial thickness and radius of the granular column). The main result was that vertical acceleration has to be included in the model if aspect ratio $a > 1$ are dealt with.

Shaltop-2d has been used successfully to simulate laboratory experiments of granular collapse (Mangeney-Castelnau et al. 2005), levees-channel formation (Mangeney et al. 2007) as well as real 3D avalanches in a natural context as it has been recently shown for large Martian landslides (Lucas & Mangeney 2007).

**Computing Resources**

We benefit of the computing facilities from the Metacentre de Données et de Calcul Parallèle of IPGP:

**CPU Server**

- Server Bi-Xeon 5160 3.00GHz
  - Mem: 3Gb
  - OS: Linux Fedora 8
  - Compiler: Intel Fortran 10.1
- IBM® e325 server bi-CPU AMD Opteron 256
  - Mem: 4Gb
  - OS: Linux RedHat Entreprise 3
  - Compiler: pgf95 - The Portland Group Inc. Fortran 90/95 compiler
Parallel Computing Cluster
- 64 x IBM x 3550 bi-CPU Intel Xeon Quad-Core E5420 (512 cores)
  - Mem: 512Gb
  - OS: RedHat EL release 5.1
  - Compiler: Intel Fortran 10.1

Time of computation could rise up to several days on the IBM e325 depending on the grid space increment and the time of the simulation. Parallelization of the code (using MPI) has been carried out by P. Stoclet at IPGP.

Softwares Used
DEM (Digital Elevation Model) processing (e.g. gridding) were performed with Surfer (Golden software). Visualization and rendering have been carry out under GMT (Wessel & Smith 1991) and MatLab.

Cases Studied
As previously mentioned, Shaltop−2d is appropriate for dry granular monophasic flow such as landslides, rock avalanches and dry sand flows. In this context, we focused our runs on the following cases even though water could be present in the real landslides:

(1) Dam break scenario from analytical solution
(2) Laboratory test of dry sand flow from USGS
(3) Shum Wan landslide, Hong Kong
(4) Fei Tsui Road landslide, Hong Kong
(5) Frank slide, Canada

DAM-BREAK SCENARIO
Description of the Scenario
This test makes it possible to calibrate the numerical modeling with analytical solution. The scenario involves movement of mass in a wide inclined channel, triggered by sudden removal of dam (Figure 2). From the analytical solution (AS) derived by Mangeney et al. (2000), profiles of the debris flow at any given time $t$ are:

$$h(t) = \begin{cases} 
H & \text{; } x \leq -x_r \\
\frac{1}{9g \cos \theta} \left( 2c_0 - \frac{x}{t} - \frac{mt}{2} \right)^2 & \text{; } -x_r < x < x_i \\
0 & \text{; } x_i \leq x
\end{cases} \quad [12]$$

with

$$m = g(\cos \theta \tan \delta - \sin \theta) \quad [13]$$

$$c_0 = \sqrt{gH \cos \theta} \quad [14]$$

$$x_i = 2c_0 t - \frac{1}{2} mt^2 \quad [15]$$
\[ x_r = c_0 t + \frac{1}{2} m t^2 \] \[ \text{[16]} \]

where \( H \) is the initial height of debris mass, \( g \) is the gravity, \( \theta \) is the slope angle and \( \delta \) is the angle of friction.

Figure 2: Initial conditions of dam-break scenario. \( H \) is the initial height of the debris mass, \( \theta \) is the slope angle and \( \delta \) is the angle of friction at channel bed.

We perform test using the following settings (Figure 3):

- Initial height of the mass: 10m
- Slope angle: 30°
- Friction angle: 25°

**Results**

*Shaltop-2d* is able predict very well the analytical solution (Figure 3).

Figure 3: Evolution of the flow height for both analytical and numerical solutions respectively named AS and NS. Using a reasonable space increment \( \Delta x \), *Shaltop-2d* is able to reproduce very well the analytical solution.
In Figure 4, we show that, for the same parameters, *Shaltop-2d* fits better with the analytical solutions than RASH3D model (Mangeney et al. 2003). For $\delta = 4^\circ$, $\theta = 5^\circ$ and $\Delta x = 20$, *Shaltop-2d* reproduces better the runout distance predicted by the analytical solution (Figure 4). The overestimation of this runout distance in the kinetic model is related to numerical diffusion. As a result, the calibration of the kinetic model in order to fit the right runout distance would lead to friction angle almost one degree higher than the real friction angle. In the simulation of real avalanches over complex topography, numerical diffusion is expected to generate larger runout distances (i.e. larger value of the fitted friction angle) using kinetic scheme or similar schemes of the first order than that obtained with *Shaltop-2d*. A series of numerical test on real topography with increasing discretization is therefore necessary if the models (i.e. the fitted friction coefficients) are to be compared on real events.

**DRY SAND BEHAVIOR WITHIN A 3D CHANNEL**

**Description of the Simulation**

Simulation of the experimental results of Iverson et al. (2004) dealing with dry granular flows
over irregular 3D channel (see Figure 5) is performed here. The basal topography has the following characteristics (Figure 5):

- Mean angle of sloping flume bed: 31.6°
- Headgate aperture width: 12 cm
- X location of headgate: 8.858 cm
- Location of urethane topographic insert: entire flume width (20cm), and covering from \( x = 9.14 \) cm to \( x = 39.12 \) cm
- The right triangular prism that spans the full width of the flume (20cm) is approximately \( h_0 = 4.35 \) cm thick (measured vertically) at \( x = 8.858 \) cm (headgate) and tapers to 0 cm thick upflume from the headgate at \( x \approx 1.9 \) cm.

We interpolate (using kriging algorithms) the DEM so as to get a space increment \( \Delta x = \Delta y = 0.24 \) cm.

![3D channel](image)

Figure 5: 3D irregular channel with the initial mass (in color)

Results obtained using a constant friction coefficient all over the topography are obviously not accurate because substrates with different roughness were used in the tank and in the irregular topography. Numerical simulation using \( \delta = 29^\circ \) all over the numerical domain is however shown for comparison (see Figure 7). The bottom part of the mass spreading is quite well reproduced by the code, whereas the top part does not fit with Iverson et al. (2004) observations. Mass staying in the tank is overestimated within the sand tank at the end of the simulation. As consequences, 15% of the bulk mass don’t participate in the spreading.

As mentioned in Iverson et al. (2004), two kind of materials are used in this 3D channel. The tank and the bottom parts are made of Formica with a bed friction angle \( \delta_f = 23.5^\circ \) and the channel part with urethane characterized by a bed friction angle \( \delta_u = 19.8^\circ \). We thus adapt our code so as to take into account the variation of the bed friction angle due to the different floor (Formica and urethane). Results are obviously significantly improved as shown in Figure 8. Mass profile is more alike experimental observations (top of Figure 6).
Figure 6: Evolution of the dry granular media across from Iverson et al. (2004)
Figure 7: Evolution of the dry granular media across an irregular 3D channel using a single friction angle $\delta = 29^\circ$ all over the numerical domain (DEM courtesy Iverson et al. 2004)
Figure 8: Evolution of the mass (left) and velocity (right) focused in the sand tank using different friction angle between Formica ($\delta_f = 23.5^\circ$) and urethane ($\delta_u = 19.8^\circ$)
THE 1995 SHUM WAN LANDSLIDE, HONGKONG

Description of the Simulation
Analysis of Shum Wan Road landslide has been carried out using Coulomb-type friction law. The initial volume in our simulation is $26 \times 10^3$ m$^3$. Using gridding algorithms (e.g. kriging), we get a 2 m grid spacing. The grid is 113 x 85 points. We perform several analysis dealing with Coulomb-type friction angle $\delta \in [16^\circ, 26^\circ]$.

![DEM of Shum Wan landslide, HK. Slide path is indicated by black lines over the topographic map. (b) Slope map calculated from the DEM.](image)

**Figure 9:** (a) DEM of Shum Wan landslide, HK. Slide path is indicated by black lines over the topographic map. (b) Slope map calculated from the DEM.

Results
The friction angle is calibrated to fit the runout distance of the landslide. Best agreement is obtained using $\delta = 18^\circ$. The mean velocity increases with a factor of two during the first 10 seconds. The flow stops at $t = 30$ s.

Table 1: Velocity evolution and statistics for Shum Wan landslide

<table>
<thead>
<tr>
<th>Time (sec.)</th>
<th>$\bar{u}$ (m/sec)</th>
<th>$U_{\text{max}}$ (m/sec)</th>
<th>$\sigma$</th>
<th>$\sigma/\bar{u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>2.39</td>
<td>11.18</td>
<td>2.71</td>
<td>1.14</td>
</tr>
<tr>
<td>16</td>
<td>5.16</td>
<td>16.53</td>
<td>4.85</td>
<td>0.94</td>
</tr>
<tr>
<td>28</td>
<td>2.56</td>
<td>13.35</td>
<td>4.17</td>
<td>1.62</td>
</tr>
</tbody>
</table>
Figure 10: Evolution of the mass (left) and velocity field (right). Results from simulation using friction angle $\delta = 18^\circ$. 
THE 1995 FEI TSUI LANDSLIDE, HONG KONG

Description of the Simulation
Analysis of Fei Tsui landslide has been carried out using Coulomb-type friction and Pouliquen friction laws. The initial volume in our simulation is $14 \times 10^3$ m$^3$ with a 1m grid spacing. The grid is 97 x 117 points. A series of simulation is performed using friction angles $\delta \in [22^\circ, 30^\circ]$.

![Figure 11](image)

Figure 11: (a) DEM of Shum Wan landslide, HK. Slide path is indicated by black lines over the topographic map. (b) Slope map from the DEM.

Results
Better results are obtained with $\delta = 26^\circ$ as it is shown in Figure 12. On the right side of Figure 12, we show the evolution of the velocity field at several times. The mean velocity increases rapidly after triggering and rise up to 70 m/s at its maximum. The flow stops at $t \approx 15$ s (see Table 2).

Finest analysis shows that, between $t = 8$ s and $t = 14$ s, the back of the mass is still under movement whereas the front does not.

As consequences, the flow spread on each side (along the road) and not in the slope direction. We thus overestimate the width of the final deposits.

Table 2: Velocity evolution and statistics for Fei Tsui landslide

<table>
<thead>
<tr>
<th>Time (sec.)</th>
<th>$\bar{u}$ (m/sec)</th>
<th>$U_{\text{max}}$ (m/sec)</th>
<th>$\sigma$</th>
<th>$\sigma/\bar{u}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.50</td>
<td>29.05</td>
<td>3.40</td>
<td>0.76</td>
</tr>
<tr>
<td>8</td>
<td>2.38</td>
<td>68.33</td>
<td>3.08</td>
<td>1.29</td>
</tr>
<tr>
<td>14</td>
<td>1.34</td>
<td>47.39</td>
<td>2.35</td>
<td>1.75</td>
</tr>
</tbody>
</table>
Figure 12: Evolution of the flow height and velocity at several times using Coulomb type friction angle $\delta = 26^\circ$. Isovalues of the thickness are represented every 1 m. On velocity held, only 1/4 vectors are displayed for a better visualization.
THE 1902 FRANK SLIDE, CANADA

Description of the Simulation

The Frank Slide involves $36 \times 10^6$ m$^3$. The grid is 201 x 201 points with 20 m space increment.

![Figure 13](image)

Figure 13: (a) DEM of Frank Slide, Canada. Slide path is indicated by black lines over the topographic map. (b) Slope map calculated from the DEM.

Results

Pirulli & Mangeney (2007) proposed a Coulomb-type friction angle $\delta = 14^\circ$ for this specific landslide. Using the same angle Shatop-2d overestimates the observed runout as it is shown in Figure 14. We get better results with $\delta = 12^\circ$ (Figure 15). Using a Pouliquen-type friction law, we get very similar results (Figure 16). Sensitivity of the numerical results to mesh size has to be performed before getting any conclusion from this comparison. Actually, the differences could only result from numerical dissipation (see discussion in the analytical solution section).

![Figure 14](image)

Figure 14: Evolution of the slide using friction angle $\delta = 14^\circ$
Figure 15: Evolution of the slide using friction angle $\delta = 12^\circ$

Figure 16: Evolution of the slide using a Pouliquen friction law where $\delta_1 = 12^\circ$, $\delta_2 = 26^\circ$ and $d = 1.5$ m
Figure 17: Velocity field at time = 10, 40 and 70 secs using Coulomb-type friction (left) and using Pouliquen-type friction law (right)
Using Coulomb-type friction or Pouliquen-type friction law have consequences on velocity field. With the second law, the flow gets faster but slows down quickly when compared with Coulomb-type friction law (see Table 3).

Table 3: Velocity evolution and statistics for both behaviors

<table>
<thead>
<tr>
<th>Time (sec.)</th>
<th>( \bar{u} ) (m/sec)</th>
<th>( U_{\text{max}} ) (m/sec)</th>
<th>( \sigma )</th>
<th>( \sigma / \bar{u} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>7.25</td>
<td>15.31</td>
<td>3.71</td>
<td>0.51</td>
</tr>
<tr>
<td>40</td>
<td>20.26</td>
<td>67.00</td>
<td>13.54</td>
<td>0.67</td>
</tr>
<tr>
<td>70</td>
<td>3.85</td>
<td>69.80</td>
<td>8.85</td>
<td>2.30</td>
</tr>
</tbody>
</table>

CONCLUSIONS

Our simulations show that, despite the accordingly non-physical empirical friction coefficient used in the numerical model, the area of the deposit is quite well reproduced in many natural cases. However, we showed that this coefficient ranges between 12° and 28° depending on the studied case. This dispersion of the friction coefficient questions the predictable power of this approach. The presence of a fluid phase, granulometry distribution, the presence of tree roots at the top of the released mass, as this is the case for Shum Wan landslide or Fei Tsui Road landslide, could explain the difference between the friction coefficient. Only calibration in the similar geological context would possibly make it possible to predict further events.

Using both friction laws (Coulomb and Pouliquen) may have inferences on velocity field. In terms of prediction, this is a key point, because of the pressure impact on the foundations.

REFERENCES


A SPH DEPTH INTEGRATED MODEL WITH PORE PRESSURE COUPLING FOR FAST LANDSLIDES AND RELATED PHENOMENA

Cedex and ETS Ingenieros de Caminos, UPM Madrid, Spain

Abstract: This paper presents a Smoothed Particle Hydrodynamics (SPH) based depth integrated model with coupling between the solid and the fluid phases which can be used to predict the behaviour of fast catastrophic landslides, avalanches and debris flows with a good compromise between accuracy and computational cost. The model performance is assessed using a set of benchmarking exercise provided by the Hong Kong Geotechnical Engineering Office.

INTRODUCTION
Modeling of fast catastrophic landslides, rock avalanches, debris flows, lahars and other similar phenomena is of paramount importance for risk management in populated areas. It is important to produce risks maps leading to urban planning legislation and mitigation measures. In order to know the scenario and the effects of the proposed engineering mitigation and protection measures, we need the information provided both by experience and by modeling. This paper deals with a simple yet effective model which can be used for prediction. The main ingredients are (i) A hierarchical set of mathematical models describing the coupled behaviour between solid skeleton and pore fluids. Here we will arrive to a coupled depth integrated model. (ii) A rheological model describing the behaviour of the fluidized fluid, and (iii) a numerical model to discretize mathematical and rheological models. In order to assess the performance of the model, a set of benchmark tests has been provided by the Geotechnical Engineering Office, Hong Kong. Of all the provided tests, we have selected a reduced number of them which we think are the most representative, in the sense that they show limitations of the proposed model. The paper is structured as follows. First of all, we introduce the mathematical model used within a general framework, showing the different alternative levels of approximation. A second section is devoted to describe the rheological models used for the benchmarks, followed by a section where we describe the discretization technique we have used. Finally, we present the selected benchmarking exercises, which include (i) a problem with an analytical solution, (ii) a landslide with a small travel distance, (iii) a rock avalanche crossing a glacier, (iv) and three debris flows.

MATHEMATICAL MODEL: A HIERARCHICAL SET OF MODELS FOR THE COUPLED BEHAVIOUR OF FLUIDIZED GEOMATERIALS
The materials found in fast landslides are mixtures where we can find soil, rocks, water and air. The behaviour of the mixture can be described using alternative approximations, which can be described in a hierarchical manner, from the more complex based on mixture theory, to the simpler depth integrated models. Here, we will describe both the model used in our computations, a coupled depth integrated model incorporating pore pressures, and the framework within which has been derived. The first mathematical model describing the coupling between solid and fluid phases was proposed by Biot (1941; 1955) for linear elastic
materials. This work was followed by further development at Swansea University, where Zienkiewicz and co-workers (1980, 1984, 1990a, 1990b, 2000) extended the theory to non-linear materials and large deformation problems. It is also worth mentioning the work of Lewis & Schrefler (1998), Coussy (1995) and de Boer (2000). It can be concluded that the geotechnical community have incorporated coupled formulations to describe the behaviour of foundations and geostuctures. Indeed, analyses of earth dams, slope failures and landslide triggering mechanisms have been carried out using such techniques during last decades.

This theoretical framework has not been applied to model the propagation of landslides until recently. We can mention here the work of Hutchinson (1986), who proposed a sliding consolidation model to predict run out of landslides, Iverson & Denlinger (2001), Pastor et al. (2002) and Quecedo et al. (2004).

**General Model**

The general model is based on the assumption that the mixture is composed of a solid phase and several fluid phases. The equations are: (i) balance of mass and (ii) balance of linear momentum for the constituents and the mixture, (iii) constitutive or rheological laws describing the material behaviour of all constituents, and (iv) kinematics relations linking velocities to rate of deformation tensors. The main problem with this approach is the computational cost, because of the number of unknowns and the difficulty of having to track all interfaces. The main advantage is its general character, as it can describe phenomena involving large relative displacements between solid and fluid phases. This model has been described in Pastor et al. (2002).

**Biot-Zienkiewicz Model**

A first simplification consists of assuming that the velocity of fluid phases relative to solid skeleton is small. In this case, the equations can be cast in terms of the displacements or velocities of solid skeleton, the velocities of the pore water relative to the skeleton and the average pore pressure of the interstitial fluids. This model is referred to as $u - \bar{p} - w$. It was introduced by Zienkiewicz & Shiomi (1984) for the case of saturated soils. The case of unsaturated soils with air at atmospheric pressure was proposed by Zienkiewicz et al. (1990b). This model consists of the three equations of balance given above, plus a suitable constitutive or rheological law providing the stress tensor and a kinematical relation relating displacements or velocities to strain or rate of deformation tensors. The main variables are (i) the velocity of solid skeleton, $v_s$, (ii) the Darcy velocity of the pore water, $w$, and the pore pressure, $p_w$. Under certain assumptions, which were analyzed for soil mechanics problems by Zienkiewicz et al. (1980), it is possible to eliminate the Darcy velocity from the model. This is the most celebrated $u - p_w$ model of Zienkiewicz, which has been widely used in geomechanics being the base of many computer codes.

**The Propagation-Consolidation Approximation**

So far we have described general models which can be applied to general problems. The analysis of landslides, due to their shape and geometrical properties, allows some interesting simplifications. First of all, we will arrive to “propagation-consolidation” models, where pore pressure dissipation takes place along the normal to the terrain surface; and next, we will describe depth integrated models, where the three dimensional problem is transformed into a two dimensional form. The propagation-consolidation model can be derived assuming that the velocity and pressure fields can be split into two components, i.e. propagation and
consolidation as

\[ v = v_0 + v_1 \text{ and } p_w = p_{w0} + p_{w1} \]  \[1\]

The equations of the propagation-consolidation model are:

\[ \rho \frac{Dv_0}{Dt} = \rho b + \text{div } \sigma \quad \text{with } \text{div } v_0 = 0 \] \[2\]

\[ \frac{Dp_w}{Dt} = \frac{\partial}{\partial x_3} \left( c_v \frac{\partial p_w}{\partial x_3} \right) \] \[3\]

where \( c_v \) is the coefficient of consolidation.

**Depth Integrated Model with Coupled Pore Pressures**

Many flow-like catastrophic landslides have average depths which are small in comparison with their length or width. In this case, it is possible to simplify the 3D propagation-consolidation model described above by integrating its equations along the vertical axis. The resulting 2D depth integrated model presents an excellent combination of accuracy and simplicity providing important information such as velocity of propagation, time to reach a particular place, depth of the flow at a certain location, etc.

Depth integrated models have been frequently used in the past to model flow-like landslides. It is worth mentioning the pioneering work of Hutter and his co-workers (Savage & Hutter, 1991; Hutter & Koch 1991), and those of Laigle and Coussot (1997), McDougall & Hungr (2004), and the authors (Pastor et al. 2002; Quecedo et al. 2004). We will use the reference system given in Figure 1 where we have depicted some magnitudes of interest which will be used in this section.

![Figure 1: Reference system and notation used in the analysis](image-url)
It is worth mentioning the difficulty of obtaining directly a Lagrangian form of the depth integrated equations, because the vertical integration is not performed in a material volume. Sometimes, it has been found to be convenient to refer to an equivalent 2D continuum having as velocities of their material points the depth integrated velocities. This cannot be considered as a Lagrangian formulation, because the moving points have no exact connection with material particles. It can be denominated either “quasi Lagrangian”, or arbitrary Lagrangian Eulerian (ALE) formulation. To derive a quasi Lagrangian formulation of the depth integrated equations, we will first introduce a “quasi material derivative” as:

$$\frac{\partial \bar{d}}{\partial t} = \partial \frac{\partial}{\partial t} + \bar{v}_j \frac{\partial}{\partial x_j}$$  \[4\]

from where we obtain the “quasi Lagrangian” form of the balance of mass, depth integrated equation as:

$$\frac{\partial \bar{h}}{\partial t} + h \frac{\partial \bar{v}_j}{\partial x_j} = e_R$$  \[5\]

where $e_R$ is the erosion rate $[L^{-1}]$

The balance of momentum equation is:

$$h \frac{\partial \bar{v}_i}{\partial t} - \frac{\partial}{\partial x_j} \left( \frac{1}{2} b_j h^2 \right) = \frac{1}{\rho} \frac{\partial}{\partial x_j} \left( h \bar{\sigma}_{ij}^* \right) + b_i h + \frac{1}{\rho} |N^b|^2 \bar{e}_R - e_R \bar{v}_i$$  \[6\]

where we have introduced the decomposition:

$$\sigma_{ij} = -\bar{p}\delta_{ij} + \sigma_{ij}^* \text{ with } \bar{p} = \frac{1}{2} \rho b_i h \text{ and } \bar{\sigma}_{ij}^* = \bar{\sigma}_{ij} + \bar{p}\delta_{ij}$$ \[7\]

The term $t_i^b$ is the i-th component of the normal stress acting on the basal surface, and $|N^b|$ is

$$|N^b| = \left( \frac{\partial Z^2}{\partial x_1} + \frac{\partial Z^2}{\partial x_2} + 1 \right)^{1/2}$$ \[8\]

where $Z$ is the height of the basal surface.

It is important to note that we have to include the effect of centripetal accelerations, which can be done in a simple manner by integrating along the vertical of the balance of momentum equation, and assuming a constant vertical acceleration given by $V^2/R$, where $V$ is the modulus of the averaged velocity and $R$ the main radius of curvature in the direction of the flow. Finally, after integrating the vertical consolidation equation in depth we arrive at:

$$\frac{\partial}{\partial t} \left( \bar{p}_s h \right) = c_i \frac{\partial p_s}{\partial x_3} \bigg|_{Z}$$ \[9\]
Next, assuming an approximation of the pore pressure as:

\[ p_w(x_1, x_2, x_3, t) = \sum_{k=1}^{N_{pw}} P_k(x_1, x_2, t) N_k(x_3) \]  

[10]

taking:

\[ N_k(x_3) = \cos\left(\frac{2k-1}{2h}\pi \left(x_3 - Z\right)\right) \quad k = 1, N_{pw} \]  

[11]

and keeping only the first term, we have:

\[ p_w(x_1, x_2, x_3, t) = P_1(x_1, x_2, t) \cos\left(\frac{\pi}{2h} \left(x_3 - Z\right)\right) \]  

[12]

from where we obtain the depth integrated equation:

\[ \frac{dP}{dt} = \frac{\pi^2}{4h^2} c \cdot P_1 \]  

[13]

which is the quasi-Lagrangian form of the vertically integrated 1D consolidation equation. It is important to note that the results obtained above depend on the rheological model chosen, from which we will obtain the basal friction and the depth integrated stress tensor, \( \tilde{\sigma}_{ij} \).

**BEHAVIOUR OF FLUIDIZED SOILS: RHEOLOGICAL MODELLING ALTERNATIVES**

When obtaining the depth integrated equations described in the preceding Section, we have lost the flow structure along the vertical, which is needed to obtain both the basal friction and the depth integrated stress tensor. A possible solution, which is widely used, consists of assuming that the flow at a given point and time, with known depth and depth averaged velocities, has the same vertical structure than a uniform, steady state flow. In the case of flow-like landslides, this model is often referred to as the infinite landslide, as it is assumed to have constant depth and move at constant velocity along a constant slope. This infinite landslide model is used to obtain necessary items in our depth integrated model.

In this work, we have used the following models: (i) a frictional fluid model, enriched with Voellmy turbulence, (ii) a Bingham fluid model with evolution of the properties depending on the entrapped solid particles, and (iii) a turbulent Manning model for water.

**Bingham Fluid**

In the case of Bingham fluids, there exists an additional difficulty, because it is not possible to obtain directly the shear stress on the bottom as a function of the averaged velocity. The expression relating the averaged velocity to the basal friction for the infinite landslide problem is:
\[ \bar{v} = \frac{\tau_y h}{6\mu} \left( 1 - \frac{\tau_y}{\tau_B} \right)^2 \left( 2 + \frac{\tau_y}{\tau_B} \right) \]  

where \( \mu \) is the viscosity; \( \tau_y \) is the yield stress; and \( \tau_B \) is the shear stress on the bottom. This expression can be transformed into:

\[ P_3(\eta) = \eta^3 - (3 + a) \eta + 2 = 0 \]  

where we have introduced \( \eta = h_p / h \) which is the ratio between the height of the constant velocity region or plug to the total height of the flow; and the non-dimensional number, \( a \), defined as:

\[ a = \frac{6\mu \bar{v}}{h \tau_y} \]  

It is first necessary to obtain the root of a third order polynomial. To decrease the computational load, several simplified formula have been proposed in the past. The authors introduced in Pastor et al. (2004) a simple method based on obtaining the second order polynomial which is the best approximation in the uniform distance sense of the third order polynomial, which is given by:

\[ P_2(\eta) = \frac{3}{2} \eta^2 - \left( \frac{57}{16} + a \right) \eta + \frac{65}{32} \]  

By knowing the non-dimensional number, \( a \), the root is obtained immediately.

Sometimes, there exists an evolution of rheological properties due to the amount of solid material eroded by the flow. The limit case is water flowing over an erodible bed, where at the beginning the fluid is water and the basal friction can be obtained by using classical formulae such as that of Manning. We have used a Bingham law with evolution where \( \tau_y = \tau_{yo} f(s) \) and \( \mu_y = \mu_{yo} f(s) \), with \( \tau_{yo} \) and \( \mu_{yo} \) being two rheological parameters describing the asymptotic limit.

The evolution function \( f(s) \) introduces a dependence on the volume fraction of soil, \( s \), given by:

\[ f(s) = 1 - \exp(-Cs) \]  

where \( C \) is an evolution constant and \( s = \left( \frac{m_s}{m_s + m_w} \right) \), which is the volume fraction of solid material.

It is important to note that the normal stress in the friction law is “effective”, and therefore we should know basal pore pressure at any instant.
Frictional Fluid

One simple yet effective model is the frictional fluid, especially in the case where it is used within the framework of coupled behaviour between soil skeleton and pore fluid. Without further additional data it is not possible to obtain the velocity distribution. This is why depth integrated models using pure frictional models cannot include information concerning depth integrated stresses, $\bar{\sigma}$. Concerning the basal friction, it is usually approximated as

$$\tau_b = -\sigma_i \tan \phi \frac{\bar{v}}{\lVert \bar{v} \rVert}$$

where $\sigma_i$ is the normal stress acting on the bottom. Sometimes, when there is a high mobility of granular particles and drag forces due to the contact with air are important, it is convenient to introduce the extra term proposed by Voellmy, which includes the correction term, $\frac{\rho g v^2}{\xi}$, where $\xi$ is the Voellmy turbulence parameter (Voellmy 1955).

In some cases, the fluidized soil flows over a basal surface made of a different material. If the friction angle between both materials, $\delta$, is smaller than the friction angle of the fluidized soil, the basal shear stress is given by:

$$\tau_b = -\rho' \tan \phi_b \frac{\bar{v}}{\lVert \bar{v} \rVert}$$

where the basal friction, $\phi_b$, is

$$\phi_b = \min (\delta, \phi)$$

This simplified model can implement the effect of pore pressure at the basal surface. In this case, the basal shear stress will be:

$$\tau_b = -\left(\sigma_i \tan \phi_b - p'_{w_b}\right) \frac{\bar{v}}{\lVert \bar{v} \rVert}$$

We can see that the effect of the pore pressure is similar to decreasing the friction angle.

Erosion

One important aspect in the behaviour of catastrophic landslides and related phenomena is the erosion. This complex phenomenon requires a rheological or constitutive behaviour of the interface, and depends on variables such as the flow structure, density, size of particles, and on how close are the effective stresses at the surface of the terrain to failure. We have used here the simple yet effective law proposed by Hungr (1995) which gives the erosion rate as $E_e = E_h \bar{v}$ where $E_e$ can be obtained directly from the initial and final volumes of the material and the distance traveled as $E_e \approx \frac{\ln(V_{final}/V_{init})}{\text{distance}}$. Units of erosion coefficient are $L^1$. 
NUMERICAL MODEL: THE SPH APPROXIMATION

To analyze the propagation of a fast landslide over a terrain, there are two main alternatives. The first is Eulerian, and is based on a structured (Finite Differences) or unstructured grid (Finite Elements and Volumes) within which the material flows. The main problem here is the need of a very fine computational mesh for both the terrain information and for the fluidized soil. Lagrangian methods allow the separation of both meshes, with an important economy of computational effort. If we combine a Lagrangian method with a mesh based discretization technique, we will find problems as soon as the mesh deforms, making necessary to use mesh refinement. As an alternative, meshless methods, which do not rely on meshes, avoid distortion problems in an elegant way. In this benchmarking exercise, we have used a meshless method referred to as the smoothed particle hydrodynamic or SPH. As in any meshless method, information is linked to moving nodes. We will describe next the method in a very succinct way. Smoothed particle hydrodynamics (SPH) is a meshless method introduced independently by Lucy (1977) and Gingold & Monaghan (1977) and firstly applied to astrophysical modelling, a domain where SPH presents important advantages over other methods. SPH is well suited for hydrodynamics, and researchers have applied it to a variety of problems, like those described in Gingold & Monaghan (1982), Monaghan & Gingold (1983), Monaghan et al. (1999), Bonet & S. Kulasegaram (2000) and Monaghan et al. (2003), just to mention a few. SPH has been also applied to model the propagation of catastrophic landslides (Bonet & Rodriguez-Paz 2005; McDougall & Hungr 2004; McDougall 2006); however in both cases, the analysis did not incorporate hydro-mechanical coupling between the solid skeleton and the pore fluid, which has been proposed by the authors (Pastor et al. 2007).

An SPH Method for Depth Integrated Equations

We will introduce a set of nodes \( \{x_k\} \) with \( K = 1..N \) and the nodal variables:

\[
\begin{align*}
    h_i & \quad \text{height of the landslide at node } I \\
    \overline{v}_i & \quad \text{depth averaged, 2D velocity} \\
    t^b_i & \quad \text{surface force vector at the bottom} \\
    \overline{\sigma}_i & \quad \text{depth averaged modified stress tensor} \\
    \overline{p}_i & \quad \text{pore pressure at the basal surface}
\end{align*}
\]

If the 2D area associated to node \( I \) is \( \Omega_I \), we will introduce for convenience:

(i) \( a \) fictitious mass, \( m_i \), moving with this node: \( m_i = \Omega_I h_i \); and

(ii) \( a \) averaged pressure term, \( \overline{p}_i \), given by: \( \overline{p}_i = \frac{1}{2} \sigma^b_i h_i^2 \)

It is important to note that \( m_i \) has no physical meaning, as when node \( I \) moves, the material contained in a column of base, \( \Omega_I \), has entered it or will leave it as the column moves with an averaged velocity which is not the same for all particles in it.

There are several possible alternatives for the equations, according to the discretized form chosen for the differential operators results. We will show those obtained with the third symmetrized forms:

---

994
\[
\frac{\partial h_i}{\partial t} = h_i \sum_j \frac{m_j}{h_j} v_{ij} \text{ grad } W_{ij} \text{ where we have introduced } v_{ij} = v_i - v_j
\]  

[23]

Alternatively, the height can be obtained, once the position of the nodes is known, as:

\[
h_i = \langle h(x_i) \rangle = \sum_j h_j \Omega_j W_{ij} = \sum_j m_j W_{ij}
\]

[24]

The discretized balance of linear momentum equation is:

\[
\frac{\partial}{\partial t} \vec{v}_i = - \sum_j m_j \left( \frac{p_i}{h_i^2} + \frac{p_j}{h_j^2} \right) \text{ grad } W_{ij} + \frac{1}{\rho} \sum_j m_j \left( \frac{\sigma_{ij}}{h_i} + \frac{\sigma_{ji}}{h_j} \right) \text{ grad } W_{ij} + b + \frac{1}{\rho h_i^2} |N_i^b|_{\theta_i}^b
\]

[25]

Finally, the SPH discretized form of the basal pore pressure dissipation is:

\[
\frac{\partial}{\partial t} P_{i\theta} = - \frac{\pi^2 c_i}{4 h_i} P_{i\theta}
\]

[26]

So far, we have discretized the equations of balance of mass, balance of momentum and pore pressure dissipation. The resulting equations are ODEs which can be integrated in time using a scheme such as Leap Frog or Runge Kutta (2nd or 4th order).

**BENCHMARKING EXERCISE**

**General Remarks**

This Section is devoted to present some of the results obtained in the modeling exercise. Due to space limitations, we have limited ourselves to the more representative cases involving some special modeling difficulties. The cases we have chosen are the following: (i) the Dam Break problem, because it provides a good idea concerning how the model is able to reproduce both convective terms and sources, (ii) the Fei Tsui Road landslide, where the propagation distance is small in comparison with other cases, (iii) the Thurwieser glacier rock avalanche, where the avalanche crosses a glacier, and the basal friction will depend on the area across which the avalanche is traveling, (iv) the 2000 Tsing Shan debris flow, where a bifurcation in the flow occurred, (v) the Tate’s Cairn exercise, where we will use the results obtained in our back prediction to predict the properties of a possible future debris flow, and finally, (vi) the Lo Wai debris flow because its complexity with water overflowing a blocked channel through the terrain and eroding it. The flow crosses roads, where a channeling effect is observed, and fluid properties change along time with the amount of eroded soil present in it.

**Benchmark 01: Dam break of a Frictional Fluid on an Inclined Plane**

The Dam Break problem has been used in the past by numerical modelers to assess the performance of the proposed models. There exist analytical solutions for several cases involving an inviscid fluid, such as the flow over dry terrain, over a “wet” plain where water
has a given depth, and over inclined slopes, and they can be found in text books such as that of Guinot (2003), for instance. The case we will model here has a solution which was proposed by Mangeney et al. (2000). Using the main variables sketched in Figure 2, the analytical solution is given by:

\[
h(t) = \begin{cases} 
\frac{1}{9g \cos \theta} \left(2c_0 - \frac{x}{t} - \frac{1}{2} mt \right)^2 & \text{if } x \leq x_L \\
0 & \text{if } x_L \leq x \leq x_R \\
0 & \text{if } x_R \leq x
\end{cases}
\]  

[27]

Figure 2: The frictional dam break problem on an inclined plane

The results obtained at times 10, 20 and 30s are plotted below in Figures 3(a), (b) and (c), showing a good general agreement between analytical and computed results.

Figure 3(a): Dam break problem: t = 10 s
This benchmark does not measure the ability of the model to reproduce shocks without oscillations. The authors have performed such test and found also good agreement between model predictions and analytical solution. We include for the case of heights at the left and right equals to 10 m and 1 m the wave profile at time 0.4 s in Figure 4. We can see that the solution consists of a rarefaction wave traveling to the left and a shock moving to the right. The model predictions agree well with the analytical solution, and are free of oscillations or other spurious phenomena.
Benchmark 05: Fei Tsui Road Landslide

This landslide, which is described in Knill and GEO (2006), occurred in August 1995 in a slope of weathered volcanic rock, grading from moderately to completely decomposed tuff. It involved 14,000 cubic meters of material. The groundwater conditions consisted of two groundwater regimes, viz. the regional groundwater table, and a perched water table. The causes are described in the report, and are a combination of a weaker material together with an increase in groundwater pressure following a prolonged heavy rainfall. The slope has an inclination of 60° and was densely vegetated. The maximum width of the mobilized mass was 90 m, and the distance travelled was 70 m. The landslide piled up some 6 m against a corner of the Baptist Church. Figure 5, taken from Knill and GEO (2006), shows a general view of the landslide.
The landslide has been modelled using a frictional fluid having an internal friction angle of 26º, following the information provided in the report. This apparent friction angle is smaller than the effective friction angle due to the existence of induced pore pressures. Taking into account the time of propagation (10s) and the mass of soil involved, we have assumed that the time of propagation is much smaller than that of pore pressure dissipation, and we have assumed undrained behaviour. We depict in Figure 6 the position of the landslide at times 1 s., 3 s, 6 s, and 9 s. Figures 7 and 8 show a comparison between field observations and model predictions. The agreement is good enough, both in the extent of the landslide and in the vertical profile providing depths.

Figure 6: Propagation of Fei Tsui Road landslide

Figure 7: Comparison between field measurements and computed results
**Benchmark 09: Thurwieser Rock Avalanche**

This case is a rock avalanche which occurred in the Central Italian Alps on the 18 September 2004. The location was the south slope of Punta Thurwieser, and it propagated through Zebrú valley. Its propagation path extended from 3500 m to 2300 m of altitude, with a travel distance of 2.9 km. The rock avalanche involved 2.2 million cubic meters. The information concerning this avalanche, including a detailed digital terrain model has been provided by Sossio & Crosta (2007). Figure 9, from Sossio & Crosta (2007), provide a general view of the avalanche and its location.

This avalanche presents several modeling difficulties, such as crossing of terrains of different materials, such as the Zebrú glacier where the basal friction is very small, and erosion of ice and snow is possible. This entrained material can melt due to the heat generated by basal friction, providing extra water, and probably originating basal pore pressures. We have used here a simple frictional model including Voellmy turbulence. Concerning erosion, we have used the law proposed by Hungr (1995). The rheological parameters chosen are: $\tan \phi = 0.39$, Voellmy coefficient $= 1,000$ m/s² and erosion coefficient $= 0.00025$ m⁻¹. The results are given in Figures 10 and 11, where we have plotted the avalanche evolution along time and the computed final extension together with the observation in the field.
Figure 9: General view of Thurwieser rock avalanche (Sossio & Crosta 2007)

Figure 10: Propagation of Thurwieser avalanche: computed results
It is important to notice that, even the extension of the avalanche is acceptable, the travel time is 8 times longer.

**Benchmark 10 The 2000 Tsing Shan Debris Flow**

This benchmarking exercise is based on the information found both in the package provided by Hong Kong Geotechnical Engineering Office and the report by King (2001). This debris flow happened on the 14th April 2000, following rains which triggered more than 50 landslides in the area. The accumulated rainfall was 160 mm. The terrain was vegetated, and consisted of colluvial boulders. One important feature of this event is the strong erosion which made the initial mass to increase from 150 to 1600 cubic meters.

Figure 12, taken from King (2001), provides two general views of the debris flow. One important aspect is the bifurcation of the flow which can be observed in the pictures.
In order to model it, we have used a frictional fluid of Voellmy type, with \( \tan \phi = 0.18 \) and a turbulent coefficient, \( \xi = 500 \text{m/s}^2 \). We have chosen the Hungr’s erosion model, using an erosion coefficient of 0.0097. The results of the simulation are given in Figures 13 and 14. It can be seen that the model is able to capture the bifurcation of the flow.
Benchmark 11: The Tate’s Cairn Exercise

In August 2005, a debris flow occurred in Tate’s Cairn, Hong Kong. Detailed analysis of the debris source revealed the existence of a damaged slope which could originate in the future more severe events. The purpose of this benchmarking exercise is, using the data of the 2005 event, to predict the consequences of a possible debris flow involving the whole damaged mass of material, as some populated areas are found further down the drainage line. Information used in this analysis is contained in the report of MGSL (2007), together with the digital terrain models provided by Hong Kong Geotechnical Engineering Office.

Concerning the source area, it was 36 m long and 22 m wide, with a maximum depth of 5.5 m approximately. The source material consisted of (i) a layer of boulder rich colluvium (or young colluvium) made of slightly sandy silty clay, about 2.9 m thick, and (ii) an old colluvium made of sandy clayey silt. Figure 15, taken from MGSL (2007), gives an overview of the 2005 debris flow event.

The rheological model chosen has been a frictional fluid of Voellmy type, with a turbulence constant, $\xi = 500 m/s^2$ and $\tan \phi = 0.3$. We have used Hungr’s erosion model, with an erosion constant of $0.0006 m^{-1}$. The results are given in figures 16 and 17, where we have depicted the evolution of the debris flow along time and a comparison between the computed results and the field observations.
Figure 15: Tate’s Cairn debris flow in 2005 (after MGSL 2007)

Figure 16: Model predictions for the 2005 event at Tate’s Cairn
Once the past event was modeled, we proceeded to analyse the characteristics of an event having a source of the whole distressed area. Figure 17 depicts the scenario, indicating the possible populated areas which could be affected.

We provide in Figures 19 and 20 the results of the forward prediction, assuming that the same material parameters are representative of fluidized soil behaviour.
Benchmark 12 The Lo Wai Debris Flow
This benchmarking exercise consists of the study of a debris flow originated from water overflowing from a catchwater which was blocked by a landslide during a rainstorm which occurred in August 2005. The information used to elaborate the model was (i) the report prepared by MGSL (2006), together with the digital terrain models provided by Hong Kong Geotechnical Engineering Office.
For the sake of completeness, we include here two figures from the MGSL (2006) report which provide a very good overall description of the scenario.

Figure 21 shows an aerial view where the catchwater has been sketched, together with the drainage lines followed by the overflowing water.

One of the main difficulties found in the analysis has been to implement in the SPH model developed by the authors an algorithm able to inject nodes in a suitable manner, together with the knowledge of the input hydrograph. To obtain it in an accurate manner, it is necessary to model a section of the channel with the blocking landslide. In this way, it would have been possible to reproduce the upstream moving wave and its arrival to both weirs. By applying a suitable weir formula, we would have been in position to obtain the hydrograph.

We have therefore done the approximation of modelling only the northern weir, as both of them join close to the catchwater channel. Of course, there will be errors in the drainage line of the southern channel. The second approximation has consisted of using a hydrograph of constant height of 0.15 m during 300 s, which is the time taken for the flow to travel the computational domain.

![Figure 21: General view of Lo Wai debris flow (after MGSL 2006)](image)
Another problem found in our computations has been the channelling effect of roads and buildings, which makes the flow to deviate from the path followed if no such elements existed. In our opinion, it would be necessary in the future to develop a “multiscale model”, where zones containing special elements would be refined. The approximation we propose here is to setup special channelling barriers in order to model these effects. Figure 2 shows the location of the barriers used in the analysis.

Concerning the rheological properties of the debris fluid, it is clear that it evolves from water to a mixture of water and the soil eroded along its path. We have assumed that the mixture behaves like a Bingham fluid, and therefore introduced an evolution law described in Section 3: \( \tau_y = \tau_{y0} f(s) \) and \( \mu_y = \mu_0 f(s) \), where \( \tau_{y0} = 150 Pa \), \( \mu_0 = 400 Pa.s \). The evolution function \( f(s) \) introduces a dependence on the volume fraction of soil, \( s \), given by

\[
f(s) = 1 - \exp(-Cs), \quad \text{with} \quad C = 0.5 \quad \text{and} \quad s = \left( \frac{m_s}{m_s + m_w} \right).
\]

This law has been complemented to take into account the behaviour of pure water when running down a vegetated slope, and the friction has been chosen as the maximum of the evolution law and that provided by Manning’s formula, using \( n = 0.06 \).

The analysis has been performed without and with channeling barriers. In the former case, the flow follows the topography, as it can be seen in Figure 24.
Figure 23: Location of the computational channelling barriers

Figure 24: Lo Wai debris flow. Computation without channeling barriers: Observation vs Computed results
In order to model the above mentioned channeling effect, we have introduced in the code these elements. The results of the computations agree now much better, as it can be seen in Figure 25.

![Figure 25: Lo Wai debris flow. Computation with channeling barriers: Observation vs Computed results](image)

**CONCLUSIONS**

Rock avalanches, flowslides, debris flows, lahars and other similar events are most complex phenomena involving complex physical mechanisms such as segregation, comminution, basal erosion, coupling with pore water, evolution of fluid properties, and thermal effects, just to mention some of them. Complete 3D models based on mixture theory and incorporating sub models for above-mentioned phenomena are still very expensive from a computational point of view. Depth integrated models provide a good combination of simplification and accuracy and can provide useful results for scientists and engineers. There exist suitable discretization techniques for depth integrated models, such as finite differences, finite elements, finite volumes or the more recent meshless methods such as the SPH model used by the authors. All of them provide accurate numerical approximations of the depth integrated equations. In the authors experience, the SPH model allows to separate the computational mesh consisting of moving nodes or particles, from the topographical mesh which can have a structured nature, simplifying very much computations on it. In the author’s experience, the computational time can be reduced up to 30 times, as compared with unstructured finite element meshes.

Finally, we provide in Figure 26 the evolution of the debris flow.
REFERENCES


**ACKNOWLEDGEMENTS**

The authors would like to express their gratitude to the organizers of the benchmarking activity as it has provided an excellent occasion to further develop and improve the SPH code. The financial support of the Spanish Ministry of Science (Project ANDROS), Ministerio de Fomento (Project MODELAD), and the Comunidad de Madrid (GATARVISA excellence network) are gratefully acknowledged.
A SET OF BENCHMARK TESTS TO ASSESS THE PERFORMANCE OF A CONTINUUM MECHANICS DEPTH-INTEGRATED MODEL

M. Pirulli and C. Scavia
Department of Structural and Geotechnical Engineering
Politecnico di Torino, Italy

Abstract: The depth averaged continuum mechanics model, RASH3D, is presented and used in this paper to numerically analyze the propagation of some cases of debris flows, landslides and laboratory experiments provided by the Hong Kong Geotechnical Engineering Office as parts of a benchmarking exercise.

INTRODUCTION
Slope failures of different type, size and velocity represent a significant threat to population, structures and infrastructures worldwide. As a consequence, modeling the runout of an unstable mass becomes fundamental. In this framework, numerical results can contribute to define the hazardous areas, estimate the intensity of the hazard (which will serve as input for risk studies), and work out the parameters for the identification of appropriate protective measures. At the same time, reliable results can help to avoid exceedingly conservative decisions regarding the development of hazardous areas.

It follows that, in recent years, administrators, planners and decision makers have become more and more interested in acknowledged numerical techniques and methods for reliable evaluation of the areas involved by the propagation of a recognized instability.

Focusing on the quantitative approach, landslide continuum dynamic models have advanced incrementally in the past three decades to the point where, when used in combination with careful engineering and geoscience judgment, first-order runout prediction is possible (McDougall et al. 2008).

The performance of the numerical code, RASH3D (Pirulli 2005, Pirulli et al. 2007), based on a single-phase continuum mechanics approach and depth averaged St. Venant equations, are here assessed through the analysis of a set of benchmark tests, provided by the Hong Kong Geotechnical Engineering Office. After a short description of the theory on which the code is based, the results obtained by its application on the proposed tests are presented and discussed.

THE RASH3D CODE
The RASH3D code (Pirulli 2005) originates from a pre-existing model (SHWCIN) based on the classical finite volume approach for solving hyperbolic systems using the concept of cell centred conservative quantities, developed by Audusse et al. (2000) and Bristeau et al. (2001) to compute Saint-Venant equations in hydraulic problems. An extension of SHWCIN for simulating dry granular flows using a kinetic scheme was initially introduced by Mangeney-Castelnau et al. (2003).
Pirulli (2005) proposed further modifications to SHWCIN to prevent observed mesh-dependency problems, permit simulation of motion across irregular 3D terrain, incorporate the influence of internal strength and allow the selection of more than one possible basal resistance relationship. The new upgraded code (RASH3D) has been validated both by laboratory tests and back analysis of real events (Pirulli et al. 2007; Pirulli & Mangeney 2008).

**Theory Description**

RASH3D treats the moving mass as a homogeneous continuum, assuming that both depth and length of the flowing mass are usually large if compared with the characteristic dimension of the particles involved in the movement. Under this assumption, it becomes possible to replace the real moving mixture of the solid and fluid phases by an “equivalent” fluid, whose rheological properties have to approximate the behaviour of the real mixture (Hungr 1995). The properties of the equivalent fluid do not correspond to those of any of the slide components.

\[ \nabla \cdot \mathbf{v} = 0 \]  \[ \rho \left( \frac{\partial \mathbf{v}}{\partial t} + \mathbf{v} \cdot \nabla \mathbf{v} \right) = -\nabla \cdot \mathbf{\sigma} + \rho \mathbf{g} \]

where \( \mathbf{v} = (v_x, v_y, v_z) \) denotes the three-dimensional velocity vector inside the avalanche in a \((x, y, z)\) coordinate system, \( \mathbf{\sigma}(x, y, z, t) \) is the Cauchy stress tensor, \( \rho \) is the mass density, and \( \mathbf{g} \) is the vector of gravitational acceleration.

Further, assuming that the vertical structure of the flow is much smaller than its characteristic length, the code integrates the balance equations in depth, obtaining the so-called depth-averaged continuum flow models (Savage & Hutter 1989). Consequently, the general system of equation to be solved becomes as follows:

\[ \begin{align*}
\frac{\partial h}{\partial t} + \frac{\partial (\bar{v}_x h)}{\partial x} + \frac{\partial (\bar{v}_y h)}{\partial y} & = 0 \\
\rho \left( \frac{\partial (\bar{v}_x h)}{\partial t} + \frac{\partial (\bar{v}_x h)}{\partial x} + \frac{\partial (\bar{v}_y h)}{\partial y} \right) & = -\frac{\partial (\sigma_{xx} h)}{\partial x} + \tau_{x(x,y)} + \rho g_x h \\
\rho \left( \frac{\partial (\bar{v}_y h)}{\partial t} + \frac{\partial (\bar{v}_x h)}{\partial x} + \frac{\partial (\bar{v}_y h)}{\partial y} \right) & = -\frac{\partial (\sigma_{yy} h)}{\partial y} + \tau_{y(x,y)} + \rho g_y h
\end{align*} \]

where \( \bar{v} = (\bar{v}_x, \bar{v}_y) \) denotes the depth-averaged flow velocity in a reference frame \((x,y,z)\) linked to the topography, \( h \) is the flow depth, \( \tau \) is the shear resistance stress in the \( x \) and \( y \) directions respectively; and \( g_x, g_y \) are the projection of the gravity vectors along the \( x \) and \( y \) directions respectively.

RASH3D allows to select the isotropy of normal stresses, \( \sigma_{xx} = \sigma_{yy} = \sigma_{zz} \), or the anisotropy of normal stresses where \( \sigma_{xx} = K \sigma_{zz} \) and \( \sigma_{yy} = K \sigma_{zz} \). The earth pressure coefficient \( K \) is defined as:
\[ K = 2 \sec^2 \phi \left( 1 + \sqrt{1 - \cos^2 \phi \sec^2 \delta} \right) \]  \[\text{[4]}\]

where \( \phi \) is the internal friction angle and \( \delta \) is the basal friction angle.

**Numerical Discretisation**

The depth-averaged St.Venant equations (Eq. [3]) are discretized in RASH3D on a general triangular grid with a finite element data structure using a particular control volume which is the median based dual cell. Fluxes of mass and momentum across cell boundaries are computed using a finite volume method (Audusse et al. 2000; Bristeau et al. 2001).

Dual cells, \( C_i \), are obtained by joining the centres of mass of the triangles surrounding each vertex, \( P_i \) (Figure 1).

![Triangular mesh and dual cell C1, C2, C3, C4. Circles denote the points under the Coulomb threshold, and stars denote the points above the Coulomb threshold (after Mangeney et al. 2003).](image)

A kinematic boundary condition is imposed on the free and bed surfaces, which specifies that mass neither enters nor leaves at the free surface or at the base.

**Rheological Models and Parameters**

Depth averaging allows a complete three dimensional description of the flow: the complex rheology of the granular material is incorporated in a single term describing the frictional stress that develops at the interface between the flowing material and the rough surface.

As for the rheological characteristics of the flowing mass, four different rheologies are implemented in RASH3D at the present time:

1. Frictional rheology, based on a constant friction angle, \( \delta \), which implies a constant ratio of the shear stress to the normal stress. Shear resistance stresses, \( \tau \), are independent of velocity.

\[ \tau_{\xi(i=x,y)} = - \left( \gamma \cos \alpha \tan \delta \right) \frac{V_i}{\|v\|} \]  \[\text{[5]}\]
where $\gamma = \text{unit weight}, \ \delta = \text{friction angle} \ \text{and} \ h = \text{flow depth}$

(2) Voellmy flow relation, which consists of a turbulent term, $v^2/g$, accounting for velocity-dependent energy losses, and a Coulomb or basal friction term for describing the stopping mechanism. The resulting basal shear stress is given by the following equation:

$$\tau_{z(i=x,y)} = \gamma \cos \alpha h \tan \delta' + \frac{\gamma v^2}{\xi} \left[ \frac{v_i}{v} \right]$$ \hspace{1cm} [6]

where $\bar{v}$ = the mean flow velocity, $\xi$ = turbulence coefficient; the others terms are similar as in Eq. [5].

(3) Quadratic rheology, where the shear resistance stress is provided by the following expression:

$$\tau_{z(i=x,y)} = - \left( \tau_y + \frac{m_{nd} v^2}{h^{1/3}} \right) v_i \left[ \frac{v_i}{v} \right] - \frac{k \eta \bar{v}}{8h}$$ \hspace{1cm} [7]

where $\tau$ is the shear stress, $\tau_y$ is the Bingham yield stress, $\eta$ is the Bingham viscosity, $n_{td}$ is the equivalent Manning’s coefficient for turbulent and dispersive shear stress components, $k$ is the flow resistance parameter.

The first and the second terms on the right hand side of equation [7] are, respectively, the yield term and the viscous term as defined in the Bingham equation. The last term represents the turbulence contribution (O’Brien et al. 1993).

(4) Pouliquen proposed in 1999 an empirical friction coefficient $\mu = \tan \delta$ that is a function of the Froude number and the thickness $h$ of the moving mass.

$$\mu = \tan \delta_1 + (\tan \delta_2 - \tan \delta_1) \exp \left( - \beta \frac{h}{d \cdot L} \frac{\sqrt{gh}}{v} \right)$$ \hspace{1cm} [8]

where the friction angle ranges between two values, $\delta_1$ and $\delta_2$, depending on the values of the velocity and thickness of the flow, $\beta$ is a constant function of the type of material (e.g. $\beta$ equal to 0.136 in case of glass beads) (Pouliquen 1999), $d$ is the mean diameter of particles involved in the movement and $L$ is a constant assumed equal to 10 (Heinrich et al. 2001).

The friction coefficient, $\mu$, is higher for small values of the thickness and high values of the velocity. The empirical relation [8] is in fact a flow rule established for steady uniform flows over inclined plane.

**BENCHMARKING**

This section presents and discusses the results obtained with RASH3D in the numerical analysis of three cases of back-analysis: Frank Slide (Alberta, Canada, 1903), Fei Tsui Road
Landslide (Hong Kong, 1995) and Tates’ Cairn Debris Flow (Hong Kong, 2005); one case of forward-analysis at Tates’ Cairn, in the same basin of the 2005 debris flow, and one laboratory experiment carried out at the EPFL (Lausanne, Switzerland).

**Frank Slide, Alberta, Canada, 1903**

The Frank Slide of 1903 was the deadliest landslide disaster of Canadian history, having destroyed a portion of the town of Frank with a loss of approximately 70 lives (McConnell & Brock 1904). It was a typical rock avalanche (for classification, see Hungr et al. 2001), involving approximately 36 million m$^3$ of fragmented limestone rock (Cruden & Krahn 1978). The rock detached from the ridge of Turtle Mountain over a front of about 700 m wide, disintegrated into fragments, descended the 800 m high slope, crossed talus aprons and glacial drift benches and covered the floodplain of Crowsnest River and the opposite hillside in a deposit of about 1.7 km wide and almost 2 km long. The deposit is approximately 18 m thick on average (Cruden & Hungr 1986) (Figure 2).

Unfortunately, information regarding movement velocity is not available. However, based on eyewitness interviews, McConnell & Brock (1904) conclude that the “time that elapsed between the first crash and complete rest did not exceed 100 seconds, and may have been somewhat less”.

The geometric data used in the present benchmarking exercise is based on a “bare earth” Digital Elevation Model (DEM) obtained by a LiDAR survey with 1 m resolution in terms of elevation, provided by the Province of Alberta Geological Survey (AGS).

![Figure 2: The Frank Slide (photo courtesy of Prof. D.M. Cruden, University of Alberta)](image)

The back analysis of the Frank Slide is carried out using both the frictional and the Voellmy rheologies.
A first set of values for rheological parameters is obtained from Hungr & Evans (1996) where the Frank event was already back-analysed with the dynamic model DAN (Hungr 1995) (Table 1).

First obtained results underestimate the propagation of the mass if a frictional rheology is adopted; while they give a good approximation of the runout if a Voellmy rheology is assumed.

Table 1: Frictional (friction angle, \( \delta \)) and Voellmy (friction angle, \( \delta' \), and turbulence coefficient, \( \xi \)) values as obtained by Hungr & Evans (1996) applying the numerical code DAN (Hungr & Evans 1996)

<table>
<thead>
<tr>
<th>Frictional</th>
<th>Voellmy</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \delta [^\circ] )</td>
<td>( \delta' [^\circ] )</td>
</tr>
<tr>
<td>16</td>
<td>5.7</td>
</tr>
</tbody>
</table>

A calibration of the frictional rheological parameters has determined good back analysis results, moving to a value of the basal friction angle (\( \delta \)) equal to 14° with an internal friction angle (\( \phi \)) set to 35° (Table 2).

In case of the Voellmy rheology, to strengthen the best fit obtained using the same values assumed by Hungr & Evans (1996), some parametric analyses were carried out but new analyses gave less satisfactory results (Table 2).

Comparing the best fit data of the two tested rheologies, it is observed that, in case of frictional rheology, the deposit tends to be short; while Voellmy rheology produces a longer deposit (Figure 3).

Since Voellmy results give a more satisfactory approximation of the real runout area, a detailed description is given in the following.

Table 2: Parametric analyses carried out with the two assumed rheologies in case of Frank slide event. The best fit parameters are indicated in bold

<table>
<thead>
<tr>
<th>Analysis</th>
<th>( \delta[^\circ] )</th>
<th>( \phi[^\circ] )</th>
<th>Analysis</th>
<th>( \delta'[^\circ] )</th>
<th>( \xi[\text{m/s}^2] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16</td>
<td>35</td>
<td>1</td>
<td>5.7</td>
<td>700</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
<td>-</td>
<td>2</td>
<td>5.7</td>
<td>500</td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>35</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>12</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td><strong>14</strong></td>
<td><strong>35</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>14</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 3: Frank Slide - Comparison between the deposit obtained by assuming Voellmy and Frictional rheologies with calibrated parameters. The red line indicates the extent of the real event (propagation and deposit).

The calculated sequence of movements described in Figure 4 and 5 visualizes the entire landslide process in terms of depth and velocity, respectively.

In Figure 4, a set of relevant depth contour diagrams show a good agreement with the field observations. It emerges that the final shape of the deposit is already defined at \( t = 60s \) when the proximal part of the mass is still moving. Frames at \( t=80s \) and \( t=100s \) are used to underline those points that are still in movement concern areas with mass depth being less than 1 meter (Figure 4, pink areas).

In Figure 5, a set of instantaneous velocity vector field diagrams shows the flow pattern and local velocity variations during the entire movement. It is evident that velocities increase rapidly with a local maximum nodal velocity of 76.60m/s and a maximum mean velocity of 48.13m/s at \( t=20s \).

At \( t=80s \) and \( t=100s \), the mean velocity has a residual value of 0.98m/s and 0.52m/s, respectively; points with instantaneous velocity higher than 0.5m/s (Figure 5, red points) concern, as already underlined, a mass with depth of less than 1 meter. It follows that it can be assumed a runout time of about 60 seconds.
Figure 4: Frank Slide – Depth contours of sliding debris using a Voellmy rheology. Contours are at 5m intervals, the red line indicates the extent of the real event (propagation and deposit).
Fei Tsui Road Landslide, Hong Kong, 1995

The Fei Tsui Road landslide took place on 13 August 1995 in Chai Wan, Hong Kong Island (Figure 6). A heavy rainstorm preceded the landslide, which took place on a cut slope excavated into the northern side of a spur of volcanic rocks that extends from the flanks of Mt Collinson in an east-north-east direction. About 14,000 m$^3$ of debris rushed down the slope, buried the road below the cut slope and caused one fatality and one injury (Chen & Lee 2000). The landslide debris disintegrated during movement, and had a travel angle of about 24° (GEO 2006). With an average vertical depth of about 15m, the landslide formed a scar with a maximum extent of 33m from the toe of the slope. The maximum horizontal travel distance of the debris was about 70m, as measured from the crest of the landslide.

The digital elevation model used in this benchmark has been built on the Geographic Information System platform based on the 1:1,000 topographic maps and spot heights. It is uniformly at 1 m-grid resolution.

The back analysis of the Fei Tsui Road Landslide is carried out using both the Frictional and the Voellmy rheologies. A first set of values for frictional rheological parameters is obtained from Chen & Lee (2000) where the Fei Tsui Road event was already back-analysed with a dynamic model. Starting values for Voellmy rheology are obtained from Hungr & Evans (1996), where it is written “in about 70% of the analysed cases, reasonable simulation of the runout was produced with the Voellmy model, with the friction angle set to 5.7° and the turbulence coefficient of 500m/s$^2$” (Table 3).

First obtained results overestimate the propagation in case of both frictional and Voellmy rheologies. A calibration of the frictional rheological parameters has determined good back analysis results moving to a value of the basal friction angle ($\delta$) equal to 26° (c.f. estimated travel angle was about 24°). With the Voellmy rheology, it has been possible to obtain a satisfactory representation of the mass runout with the combination of parameters $\delta'$= 25° and $\xi=1000$m/s$^2$ (Table 4).

In this case, it is observed that the frictional rheology and the Voellmy rheology produces approximately the same runout, deposit shape and velocities. It seems that, in this case, $\xi=1000$m/s$^2$ has about the same effect where the difference between $\delta$ and $\delta'$ is small. The results obtained with the Voellmy rheology are discussed in the following.

The calculated sequence of movements described in Figures 7 and 8 visualizes the entire landslide process in terms of depth and velocity, respectively. In Figure 7, a series of relevant depth contour diagrams shows a good agreement with the field observations. It emerges that the final shape of the deposit is already reached at $t=15$s. In Figure 8, a series of instantaneous velocity vector field diagrams shows the flow pattern and local velocity variations during the entire movement. It is evident that velocities increase rapidly with a local maximum nodal velocity of 11.42m/s at $t=1.5$s and a maximum mean velocity of 5.33m/s at $t=2.5$s. At $t=15$s and $t=20$s, the mean velocity has a residual value of 0.005m/s and 0.003m/s, respectively; and points with instantaneous velocity higher than 0.05m/s (Figure 8, red points) mainly concern sectors in the boundary of the triggering area. It is then considered a runout time of about 15 seconds.
Figure 5: Frank Slide – Instantaneous velocity vector fields (Voellmy rheology). Points with velocity higher than 0.5m/s are shown in red.
Figure 6: The Fei Tsui Road Landslide – Site layout plan (after GEO 2006)

Table 3: Frictional and Voellmy values as obtained by Chen & Lee (2000) and Hungr & Evans (1996), respectively

<table>
<thead>
<tr>
<th>Frictional</th>
<th>Voellmy</th>
</tr>
</thead>
<tbody>
<tr>
<td>δ [°]</td>
<td>φ [°]</td>
</tr>
<tr>
<td>22</td>
<td>35</td>
</tr>
</tbody>
</table>

Table 4: Parametric analyses carried out with the two assumed rheologies in case of Fei Tsui Road event. The best fit parameters are indicated in bold

<table>
<thead>
<tr>
<th>Frictional</th>
<th>Voellmy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analysis</td>
<td>δ [°]</td>
</tr>
<tr>
<td>1</td>
<td>22</td>
</tr>
<tr>
<td>2</td>
<td>22</td>
</tr>
<tr>
<td>3</td>
<td>24</td>
</tr>
<tr>
<td>4</td>
<td>24</td>
</tr>
<tr>
<td>5</td>
<td>26</td>
</tr>
<tr>
<td>6</td>
<td>27</td>
</tr>
<tr>
<td>7</td>
<td>28</td>
</tr>
<tr>
<td>8</td>
<td>28</td>
</tr>
</tbody>
</table>
Figure 7: Fei Tsui Road landslide – Depth contours of sliding debris. Contours are at 1m intervals, the red line indicates the extent of the real event (deposit).
Figure 8: Fei Tsui Road landslide – Instantaneous velocity vector fields. Points with velocity higher than 0.05 m/s are shown in red.

A cross section of the landslide is also given in Figure 9. Numerical results give a good correspondence between the real and the calculated maximum travel distance of the debris. As to the real depth distribution profile, the calculated profile has a greater depth in the lower part and a greater depth in the upper part.
Figure 9: Fei Tsui Road landslide – Section through the landslide (blue line in plan). Plan: the red line indicates the extent of the real event (deposit); the black contours are depth contours of model prediction (deposit). Section: the red points indicate the real event profile (deposit); the black points indicate the profile as defined by the model prediction (deposit) and the green points represent the slope profile.

Tates’ Cairn Debris Flow, Hong Kong, 2005
In the early morning of 22 August 2005, following heavy rainfall on 19 and 20 August 2005, a debris flow was reported to have occurred on a natural hillside about 200 m to the north of Tate’s Ridge and about 500 m south of Kwun Ping Road, Kwun Yam Shan, Hong Kong (Figure 10).

The failure involved colluvium of up to 5 m thick with a total volume of about 2,350 m³. A large portion of the displaced material (about 1,350 m³) remained within the source area as intact rafts separated by a series of stepped tension cracks. The rest of the detached mass (about 1000 m³) entered the ephemeral drainage line below, and developed into a channelised debris flow (Figure 10) (MGS 2007).
The landslide debris travelled a total distance of about 330 m down the drainage line and came to rest at two distinct boulder dams within the drainage line. The difference in elevation between the landslide source and the end of the debris trail was approximately 138 m, with a travel angle of about 24° (Wong & Ho 1996).

The digital elevation model used in this exercise has been built on the Geographic Information System platform from the published 1:1,000 topographic maps and spot heights. It is uniformly at 5 m-grid resolution.

![Figure 10: Site layout map of the Tates’ Cairn Debris Flow (after MGS 2007)](image)

The back analysis of the Tates’ Cairn Debris Flow is carried out using the frictional, the Voellmy and the quadratic rheologies. The first set of rheological parameters are obtained from the field estimated travel angle reached by the debris in Tates’ Cairn event (Table 5 – No. 1) for the frictional rheology; from the back-analysis of historical debris flows in Hong Kong (even if referred to event of detachment of 10000m³ material at sources) (Table 5 – Nos. 2-3-4) for the Voellmy rheology; and from the back-analysis of South Italy debris flows (Table 5 – Nos. 5-6) for the quadratic rheology.

First obtained results overestimate the real propagation in case of frictional, Voellmy and quadratic rheologies when Nos. 1, 2, 3, 5 set of parameters (Table 5) are adopted in the analyses; while in the case of quadratic rheology, No. 6 set of parameters underestimates the propagation. The results show a good approximation of the maximum runout distance when No. 4 set of parameter are adopted in the analysis in the case of Voellmy rheology.

Table 5: Frictional, Voellmy and quadratic values based on literature and on the back analyses of historical debris flows in Hong Kong, respectively

<table>
<thead>
<tr>
<th>N°</th>
<th>Frictional</th>
<th>Voellmy</th>
<th>Quadratic</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>δ [°]</td>
<td>φ [°]</td>
<td>τy [kPa]</td>
</tr>
<tr>
<td>1</td>
<td>23</td>
<td>-</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>500</td>
<td>1000</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>

<p>| | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>25</td>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>
A calibration of the frictional rheological parameters has determined good back analysis results moving to a value of the basal friction angle equal to 27° (field estimated travel angle was of about 23°). In case of quadratic rheology, it was possible to obtain a satisfactory representation of the mass runout by assuming $\tau_y = 1.2\text{ kPa}$, $\eta = 40\text{ Pa}\cdot\text{s}$ and $n = 0.03$; while the best fit obtained with the Voellmy rheology is defined, as already mentioned, with $\delta' = 25^\circ$ and $\xi = 1000\text{ m/s}^2$ (Table 6). The obtained results are discussed in the following:

Table 6: Parametric analyses carried out with the three assumed rheologies in case of Tates’ Cairn debris flow. The best fit parameters are indicated in bold

<table>
<thead>
<tr>
<th>N°</th>
<th>$\delta$ [°]</th>
<th>$\phi$ [°]</th>
<th>N°</th>
<th>$\delta'$ [°]</th>
<th>$\xi$ [m/s$^2$]</th>
<th>N°</th>
<th>$\tau_y$ [kPa]</th>
<th>$\eta$ [Pa•s]</th>
<th>$n$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23</td>
<td>-</td>
<td>1</td>
<td>8</td>
<td>500</td>
<td>1</td>
<td>0.7</td>
<td>10</td>
<td>0.04</td>
</tr>
<tr>
<td>2</td>
<td>23</td>
<td>35</td>
<td>2</td>
<td>15</td>
<td>1000</td>
<td>2</td>
<td>1.5</td>
<td>10</td>
<td>0.04</td>
</tr>
<tr>
<td>3</td>
<td>24</td>
<td>35</td>
<td>3</td>
<td>25</td>
<td>1000</td>
<td>3</td>
<td>1.5</td>
<td>10</td>
<td>0.02</td>
</tr>
<tr>
<td>4</td>
<td>26</td>
<td>-</td>
<td>4</td>
<td>20</td>
<td>1000</td>
<td>4</td>
<td>2</td>
<td>10</td>
<td>0.02</td>
</tr>
<tr>
<td>5</td>
<td>28</td>
<td>-</td>
<td>5</td>
<td>15</td>
<td>1000</td>
<td>5</td>
<td>1.5</td>
<td>15</td>
<td>0.04</td>
</tr>
<tr>
<td>6</td>
<td>27</td>
<td>-</td>
<td>6</td>
<td>1.2</td>
<td>40</td>
<td>6</td>
<td>1.2</td>
<td>40</td>
<td>0.03</td>
</tr>
<tr>
<td>7</td>
<td>27</td>
<td>35</td>
<td>7</td>
<td>1.2</td>
<td>40</td>
<td>7</td>
<td>1.2</td>
<td>35</td>
<td>0.02</td>
</tr>
</tbody>
</table>

In comparison with the frictional rheology, the Voellmy rheology introduces an additional parameter to be calibrated but, as the frictional rheology, it is not able to simulate the deposition along the path. The final deposit of the mass is still concentrated in the final part of the propagation path. The last tested rheology is the quadratic rheology. It adds a further parameter to be calibrated but it allows to better approximate the deposition of material along the path of propagation.

The calculated sequence of movements described in Figures 11, 12 and 13 visualizes the entire landslide process in terms of depth using a frictional, a Voellmy and a quadratic rheology, respectively. In each figure, a series of relevant depth contour diagrams show a good agreement with the field maximum runout distance of the moving mass.

Comparing the results, it can be immediately observed that the final deposit generates at different times. In case of frictional and Voellmy rheologies, about 250 seconds are necessary; while the quadratic rheology defines the deposit in only 50 seconds. The shape of the deposit obtained with the Voellmy rheology is similar to that obtained with the frictional rheology in term of longitudinal extension, and to that obtained with the quadratic rheology in term of lateral expansion. As already mentioned, in comparison with the others adopted rheologies, the quadratic rheology allows the deposition of material along the path of propagation.

A fundamental difference emerges among the maximum depth measured in the final deposits; the maximum value is reached in the proximal part of the frictional deposit (1.35m), the intermediate is distributed all along the longitudinal extension of the Voellmy deposit (0.95m) and the minimum is in the distal part of the quadratic deposit (0.30m). None of the three rheologies reproduces the deposit in an exact way. The Voellmy and the frictional rheologies are more correct in terms of maximum depth and not in terms of deposit shape; while the quadratic rheology is more correct in terms of deposit shape and not in terms of maximum depth.
Figure 11: Tates’ Cairn debris flow 2005 – Depth contours of sliding debris (frictional rheology). The red line indicates the extent of the real event (propagation and deposit). Contours are at 0.2m intervals.
Figure 12: Tates’ Cairn debris flow 2005 – Depth contours of sliding debris (Voellmy rheology). The red line indicates the extent of the real event (propagation and deposit). Contours are at 0.2m intervals.

Figure 13: Tates’ Cairn debris flow 2005 – Depth contours of sliding debris (quadratic rheology). The red line indicates the extent of the real event (propagation and deposit). Contours are at 0.05m intervals.
The calculated sequence of local velocity variations during the entire movement described in Figures 14, 15 and 16 visualizes the entire landslide process in terms of instantaneous velocity vector field using a frictional, a Voellmy and a quadratic rheology, respectively. It is evident that velocities increase rapidly with a local maximum nodal velocity of 13.08 m/s and a maximum mean velocity of 8.56 m/s at $t=20s$ with a frictional rheology; a local maximum nodal velocity of 6.52 m/s and a maximum mean velocity of 4.48 m/s at $t=10s$ with a Voellmy rheology; and a local maximum nodal velocity of 13.64 m/s and a maximum mean velocity of 6.65 m/s at $t=10s$ with a quadratic rheology. The mean velocity has a residual value included in the range 0.03 – 0.05 m/s.

Figure 14: Tates’ Cairn debris flow 2005 – Instantaneous velocity vector fields (frictional rheology)
Figure 15: Tates’ Cairn debris flow 2005 – Instantaneous velocity vector fields (Voellmy rheology)

Figure 16: Tates’ Cairn debris flow 2005 – Instantaneous velocity vector fields (quadratic rheology)
Tates’ Cairn Debris Flow, Hong Kong Forward Prediction

A detailed inspection of the hillside above the August 2005 landslide in March 2006 revealed an extensive system of tension cracks (Figure 10). These tension cracks define an area of distressed hillside with an estimated volume of 10,000 m³ located on the southeast side of the August 2005 landslide source area. The possible toe of this distressed hillside lies at an elevation between 430m and 440m above sea level and has an average inclination of about 20°-25° (Figure 10) (MGS 2007). The digital elevation model used in this benchmark has been built on the Geographic Information System platform from the published 1:1,000 topographic maps and spot heights. It is uniformly at 5 m-grid resolution.

The prediction of the Tates’ Cairn debris flow is carried out with the calibrated parameter of the frictional, the Voellmy and the quadratic rheologies as obtained in the back analysis carried out for the event that took place in Tates’ Cairn in 2005 and described in the previous section. Adopted parameters are quoted in Table 7 and obtained results are described in the following:

Table 7: Frictional, Voellmy and quadratic parameters calibrated on the Tates’ 2005

<table>
<thead>
<tr>
<th>Frictional</th>
<th>Voellmy</th>
<th>Quadratic</th>
</tr>
</thead>
<tbody>
<tr>
<td>δ [°]</td>
<td>φ [°]</td>
<td>δ' [°]</td>
</tr>
<tr>
<td>27</td>
<td>-</td>
<td>25</td>
</tr>
</tbody>
</table>

Analysis of the Numerical Results

Using the calibrated rheological parameters, a large difference emerges among the deposits obtained with the frictional, the Voellmy and the quadratic rheology (Figure 17). Results obtained adopting a frictional or a Voellmy rheology define a similar maximum travel distance. With the frictional rheology, a large part of the mass remains within the triggering area. The propagation determined with the quadratic rheology amazingly differs from the others.

The particularity of obtained results has induced an in-depth investigation that has been carried out following two different directions:

1. Analysis of volume effect
2. Comparison with DAN code (Hungr 1995) results

First adopted calibrated rheological parameters are based on the back analysis of a mass that is 10 times smaller (1·10³m³) than that here investigated (10·10³m³). To understand the possible influence of the difference in volume, the depth of the investigated volume is cut by 90% and a volume of about 1·10³m³, with unchanged basal area, is obtained. The propagation of the new volume (1·10³m³) is then analysed with the same calibrated quadratic rheological parameters used in the previous forward-analyses. Results obtained are represented in Figure 18 where it clearly emerges that the quadratic rheology is largely influenced by the volume (see the corresponding quadratic propagation in Figure 17).

Coming back to the initial analysis of propagation (10·10³m³), it is not possible to conclude which rheology gives the right propagation. It would be necessary to further investigate this aspect and couple a back analysis and a prediction having a similar volume.
Figure 17: Tates’ Cairn debris flow forward prediction – Depth contours of sliding debris. Comparison among Frictional (contours at 0.4m intervals), Voellmy (contours at 0.4m intervals) and Quadratic (contours at 0.1m intervals) rheologies with parameters calibrated on the 2005 Tates’ Cairn event. The red line indicates the extent of the 2005 Tates’ Cairn real event (propagation and deposit).

Figure 18: Tates’ Cairn debris flow forward prediction – Depth contours of sliding debris assuming a volume of 1,000m$^3$ and the quadratic parameters calibrated on the 2005 Tates’ Cairn event. Contours are at 0.1m intervals. The red line indicates the extent of the 2005 Tates’ Cairn real event (propagation and deposit).
To further validate the RASH3D results, a comparison of the propagation given with a frictional, a Voellmy and a Bingham rheology is also carried out with the DAN code (Hungr 1995) (Figure 19).

Figure 19: Example of results obtainable with the DAN code (Hungr 1995) assuming a Frictional in (b) a Voellmy in (c) and a Bingham rheology in (d); (1) V=600m$^3$ on a short triggering area, (2) V=2,400m$^3$ on a short triggering area, (3) V=2,400m$^3$ on a long triggering area.

Since DAN does not implement the quadratic rheology, the Bingham rheology is adopted in place. The comparability of the two rheologies is justified by the presence of the Bingham yield stress, $\tau_y$, and the Bingham viscosity, $\eta$, in both the rheologies. Three different initial configurations are investigated: a small volume of about 600m$^3$ (Figure 19 - 1a), a large volume of about 2400m$^3$ concentrated on the same area as the small volume (Figure 19 – 2a) or on a more extended area, generating lower depths of the triggering volume (Figure 19 – 3a).

Figures 19(1b), 19(1c) and 19(1d) represent the propagation of the small volume with the calibrated rheological parameters in case of frictional, Voellmy and Bingham rheology, respectively. The maximum distance reached by the mass is the same with the three rheologies. The final deposit is highly concentrated in the distal part of the runout profile with the frictional and the Voellmy rheologies; while the mass deposits along the whole propagation path with the Bingham rheology.

Figures 19(2b), 19(2c) and 19(2d) represent the propagation of the large concentrated volume with the rheological parameter values used for the small volume in case of frictional, Voellmy and Bingham rheology, respectively. Results obtained from a frictional or a Voellmy rheology define a similar maximum propagation distance that is a little longer than that defined for the small volume. The propagation determined with the Bingham rheology strongly differs from the others and the mass covers a very long distance. Figures 19(3b), 19(3c) and 19(3d)
represent the propagation of the large extended volume with the rheological parameter values used for the small volume in case of frictional, Voellmy and Bingham rheology, respectively. The obtained results are similar to that obtained in case of the large concentrated volume.

It is again possible to conclude that the Bingham rheology, as the quadratic rheology, is largely influenced by the volume. Furthermore, it emerges that, volumes being equal, the spatial distribution of the mass is not important.

**Deflected Sand Flow Experiment, EPFL**

A laboratory experiment was carried out at the EPFL. A volume of 30,000 cm$^3$ of Hostun sand was released from a box onto a plane with 37.5° slope and was deflected by a dike oriented obliquely to the flow direction. The back analysis of the deflected sand flow laboratory test is carried out assuming a frictional rheology. Starting values for rheological parameters are those suggested by the EPFL and quoted in Table 8.

<table>
<thead>
<tr>
<th>Frictional</th>
<th>( \delta ) [°]</th>
<th>( \phi ) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>32</td>
<td>34</td>
</tr>
</tbody>
</table>

Adopting these values, the results obtained with RASH3D overestimate the mass propagation. A calibration of the rheological parameters has given good back analysis results moving to a value of the basal friction angle (\( \delta \)) equal to 35°. Superimposing the experimental final deposit shape on the model deposit, a good approximation is obtained (Figure 20), even though it emerges a higher numerical deposition along the final part of the plane of propagation and in the external part of the deflector plate.

![Figure 20: Depth contours of sliding debris. Contours are at 0.5cm intervals. The red line indicates the experimental final deposit.](image)

The calculated sequence of movements described in Figures 21 and 22 visualizes the entire landslide process in terms of both depth and velocity. In Figure 21, a series of relevant depth contour diagrams show a rather good agreement with the laboratory observations.
Figure 21: Depth contours of sliding debris. Contours are at 0.5cm intervals. The red line indicates the numerical mass propagation.

In Figure 22, a series of instantaneous velocity vector field diagrams shows the flow pattern and local velocity variations during the entire movement. It is evident that velocities increase rapidly with a local maximum nodal velocity of 1.86m/s at $t=0.42s$ and a maximum mean velocity of 0.97m/s at $t=0.56s$. At $t=3.36s$, the mean velocity has a residual value of 0.005m/s and a local maximum nodal velocity of 0.36m/s.
CONCLUSIONS

The RASH3D code has been used in the present paper to run three back-analysis of full scale events: Frank Slide (Alberta, Canada, 1903), Fei Tsui Road Landslide (Hong Kong, 1995) and Tates’ Cairn Debris Flow (Hong Kong, 2005); one forward-analysis at Tates’ Cairn, in the same basin of the 2005 Tate’s Cairn debris flow, and one laboratory experiment carried out at the EPFL (Lausanne, Switzerland). The above mentioned cases have been analysed adopting three different rheologies (frictional, Voellmy, quadratic).
Both the Frank Slide and the Fei Tsui Road landslides have been simulated in a satisfactory way using a Voellmy rheology. The Tates’ Cairn back-analysis and forward-analysis have allowed to underline the importance of volume when parameters calibrated through back-analysis are used for prediction purposes in a case having a volume different from that of the back-analyzed event. In this framework, the results obtained with a frictional, a Voellmy and a quadratic rheology underline that the quadratic rheology is largely influenced by volume.

The laboratory experiment has been reproduced in a rather satisfactory way with a simple frictional rheology. These results are encouraging and justify the interest that continuum models have received in the previous years. At the same time, they have underlined the importance of choice of rheology since not all the events can be simulated with the same rheology. Obviously, experience and critical examination by the user plays an important role in evaluating numerical analyses reliability.

REFERENCES
Civil Engineering and Development Department, The Government of the Hong Kong Special Administrative Region, 138.

ACKNOWLEDGEMENTS
The authors wish to thank the Review Committee of the Benchmarking Exercise for their invitation to participate in the benchmarking exercise and for having provided this interesting and fruitful opportunity of testing and comparing numerical codes from worldwide.
LANDSLIDE DETACHMENT MECHANISMS: AN OVERVIEW OF THEIR MECHANICAL MODELS

Rainer Poisel and Alexander Preh
Institute for Engineering Geology, Vienna University of Technology

Abstract: A catalogue of possible landslide detachment mechanisms, taking into account the geological setting and the geometry of the slope, the joint structure, the habitus of the rock blocks, as well as the mechanical behaviour of the rocks and of the rock mass (deformation and strength parameters), is presented. Its aim is to give geologists as well as engineers the opportunity to compare phenomena in the field and phenomena belonging to particular mechanisms and to find the mechanism occurring. The presented catalogue of initial rock slope failure mechanisms only comprises mechanisms having a clearly defined mechanical model.

INTRODUCTION
A catalogue of possible landslide detachment mechanisms (Figure 1) is presented giving geologists as well as engineers the possibility to compare phenomena in the field and phenomena belonging to particular mechanisms in order to identify the current mechanism in a special case and to apply the adequate mechanical model. This catalogue takes into account the geological setting and the geometry of the slope, the joint structure, the habitus of the rock blocks, as well as the mechanical behaviour of the rocks and of the rock mass (deformation and strength parameters). The possible landslide detachment mechanism must be the basis for:

- Monitoring (which quantity has to be measured? where? Kovari (1990)) and interpretation of monitoring results;
- Modelling and analyses (only a mechanism embedded in a model can be the result of an analysis. There is no model at present comprising all possible mechanisms);
- Risk assessment;
- Design of measures for decreasing instability and for warning.

Many classifications of rock slope failure mechanisms do not distinguish between failure or detachment mechanism and the possible run out (e.g. rockfall, rock slide, rock avalanche; Hungr & Evans (2004)). As the failure mechanism (Poisel & Preh 2004) influences the stability, the run out (Poisel & Roth 2004) of a failure affects the danger for settlements etc. An ideal model should therefore simulate both the failure mechanism and the run out. At the moment, we do not have such a model.

LANDSLIDE DETACHMENT MECHANISMS

Falling of Rock Blocks
“Falling” is a frequently used term in many rockslide classifications. However, the examples shown in these classifications have very little to do with a real fall. They are mostly slides turning into a fall in later phases. The Block Theory by Goodman & Shi (1985) shows that
“falling” as an initial failure mechanism of a rock slope can be the result of an overhang only. Therefore it only occurs in massive rocks with clearly defined joints.

**Sliding of a Rock Block on a Single or on Two Discontinuities**

Most probably translational sliding of a rock block on an inclined discontinuity is the initial failure mechanism of a rock slope. It is not common knowledge, however, that commercial programs analysing sliding of rock blocks on a single or on two discontinuities give false results when investigating cases with large forces pulling out of the slope (e.g. anchorage of tautline cableway). Only Block Theory by Goodman & Shi (1985) can analyse such cases in a correct way.

**Sliding of Several Rock Blocks on a Polygonal Sliding Plane**

Sliding of a rock mass on a polygonal sliding plane is possible only when antithetic fractures (Mandl 1988) exist or develop during movements of the slope, making shear displacements between the blocks possible. The model best suited for analysing this mechanism is the kinematical element method (Gussmann 1988). UDEC is also able to simulate such mechanisms (Figure 2; Zettler et al. 1999a; 1999b).

**Rock Slumping**

Rock slumping is a characteristic mode of backward rotation of rock blocks (Kieffer 2003) similar to a ladder leaned too gently against a wall. As with toppling failures, rock slumps involve load interaction between steeply inclined columns that are rotationally unstable, and occur when pure sliding along the discontinuities is inadmissible. Kieffer (1998) gave a limit equilibrium analysis for this mechanism. Discrete Element Codes (e.g. UDEC, DDA by Shi & Goodman (1984)) can also model this mechanism effectively.

**Rotational Sliding of a Fractional Body on a Shelly, Newly Formed Sliding Surface (Circular Failure)**

Though rock slope failures are controlled by geological features (mostly some few discontinuities) in general, a circular failure like in soil can occur in rock masses of low strength, e.g. heavily fractured rock, when block dimensions are much smaller compared to slope height. As the geometry of circular failures in soft or heavily fractured rock is similar to that in soil, the stability assessment methods used for soil slope failures (e.g. Bishop 1955; Janbu 1954) can also be applied to circular failures of rock slopes. It is important to note that the main scarp of a rotational slide is a normal fault (Figure 3) and not a tension crack.
Falling of rock blocks  
R.E. GOODMAN &  
G.-H. SHI (1985)

Sliding of a rock block  
on a single or on two  
discontinuities  
Translational sliding  
R.E. GOODMAN &  
G.-H. SHI (1985)

Sliding of several rock blocks  
on a polygonal sliding plane  
P. GUSSMANN (1988)

Rock slumping  
Backward rotation  
of rock blocks  
D.S. KIEFFER (1998)

Sliding of a fractional body on a  
shelly, newly formed sliding  
surface  
Rotational sliding;  
mainly in rock masses of low  
strength, e.g. heavily fractured  
rock; block dimensions << slope  
height  
A.W. BISHOP (1955)

Translational or rotational  
descent of tower- or slab-  
shaped blocks of competent  
rock upon an incompetent base  
"Hard on soft"  
R. POISEL &  
W. EPPENSTEINER (1988)

Figure 1(a): Landslide detachment mechanisms and their mechanical models, Part 1
Rotation of single rock blocks
E. g. rotation of a rock block on a discontinuity due to eccentric bearing or partial yielding of bearing, slumping of one rock block
W. WITTKE (1990)

Buckling of column– or slab-shaped rock blocks
Column- or slab thickness << slope height
D. S. CAVERS (1981)

Flexural toppling
Bending of column- or slab-shaped rock blocks like cantilever beams
M. HITTINGER & R.E GOODMAN (1978)

Toppling of column– or slab-shaped rock blocks
Forward rotation similar to dominos; mainly when joint strength is low and rock block strength is high
R.E GOODMAN & J.W. BRAY (1976)

Slope creep
Continuously decreasing creep of rock mass downslope with increasing depth (mainly in rock masses of low strength, e.g. shales, phyllites)
O.C. ZIENKIEWICZ, C. HUMPHESON & R.W. LEWIS (1975)

Kink band slumping
S-shaped deformation of rock lamellae due to slope creep

Figure 1(b): Landslide detachment mechanisms and their mechanical models, Part 2
Figure 2: Sliding of several rock blocks on a polygonal sliding plane modelled using UDEC

Figure 3: Hope slide: The main scarp is a normal fault
Translational or Rotational Descent of Tower– or Slab-Shaped Blocks of Competent Rock upon an Incompetent Base (“Hard on Soft”)
The system of hard, competent rock (e.g. massive limestone) lying on a soft, incompetent, ductile base (e.g. phyllites, slate) is a case appearing more often than generally believed. Due to the squeezing out and yielding of the incompetent base material, the competent rock is subjected to tensile stresses, therefore fractured intensively and thus shows disintegration into huge blocks (Figure 4; Poisel & Eppensteiner 1988).

Generally these blocks may (Figure 5):

1. slide downhill translatic and upright,
2. form a rotational slide together with the moving base material (internal, backward rotation) or
3. topple downhill (external rotation; most dangerous case leading to sudden rock avalanches).

This mechanism can reach much deeper into the slope than other mechanisms. Modelling this mechanism is possible using FLAC or PFC (Figure 6).

Figure 4: Disintegration of competent rock lying on an incompetent base into tower- or slab- shaped rock blocks

(a) (b) (c)

Figure 5: Movements of tower- or slab shaped rock blocks at the edges of competent rock lying on an incompetent base. (a) Translational and upright sliding down, (b) circular failure in the base, and (c) toppling
Figure 6: Squeezing out of the incompetent base material and fracturing of the competent rock above simulated by PFC

**Rotation of Single Rock Blocks**
Rotation of rock blocks around an axis horizontal and parallel to the slope surface is part of many initial failure mechanism of rock slopes (e.g. rock slumping, toppling). However, rotation of single rock blocks with a rotation axis not horizontal and/or not parallel to the slope surface, such as a torsional failure according to Goodman (2003) or slumping of just one rock block, are special cases, which are not trivial to analyse. Physical models very often help a great deal in understanding such cases. Wittke (1990) gave an overview of such cases and formulae for analysing this mechanism.

**Buckling of Column– or Slab-Shaped Rock Blocks**
Buckling failure can occur in slopes built up by rock columns or rock slabs which are thin compared to the slope height. Eulerian buckling formulae by Cavers (1981) give extremely conservative results in general, because Cavers (1981) estimated the buckling length much too long. Numerical investigations using PFC by Preh (2004) (Figure 7) showed that the buckling length is about one quarter of the total slope length and that the Eulerian buckling formulae by Cavers (1981) overestimate the critical load for slopes which are almost vertical. Furthermore, they underestimate the critical load for lower inclinations, taking into account the correct buckling length. The almost vertical slopes are therefore less stable than the Cavers model predicts, taking into account the correct buckling length; and the slopes with lower inclinations are more stable than the Cavers model predicts, taking into account the correct buckling length.

**Toppling**

**Flexural Toppling**
Flexural toppling (Goodman & Bray 1976) is the result of the overturning and cantilever beam-like bending of rock blocks formed by joints (e.g. schistosity, bedding) dipping into the slope. The stresses resulting from cantilever beam-like bending may cause a second set of joints normal to the first one. A typical feature of flexural toppling as well as block toppling is the sawtooth pattern of the slope surface.

**Toppling of Column– or Slab-Shaped Rock Blocks (Block Toppling)**
When the second set of joints is more intense, block toppling takes place, which is a forward
rotation of rock blocks similar to dominos; mainly when joint strength is low and rock block strength is high (Figure 8).

Flexural as well as block toppling can be effectively modelled numerically by the discrete element codes UDEC (Universal Distinct Element Code) and 3DEC from Itasca.

**Chevron Toppling**
As a consequence of progressive failure in the joints dipping out of the slope, block toppling may result in a sliding failure after a certain amount of toppling. This mechanism was called chevron toppling by Cruden et al. (1993).

**3D-Effects**
Goodman (1980) pointed out that toppling can occur only if the layers strike nearly parallel to the strike of the slope within 30°. Numerical investigations using 3DEC by Wollinger (2003) showed that toppling is possible if the strike difference is up to 40° (Figure 9). However, the factor of safety increases with increasing deviation of the strike of the layers and the strike of the slope.

![Figure 7: Buckling failure of a slab shaped rock block modelled using PFC](image)

![Figure 8: Typical of toppling failures](image)
There is no difference between toppling and slope creep in principle (displacement rate distributions in Figures 8 and 12), because reducing the spacing of the joints dipping into the slope means a change from toppling to slope creep. However, toppling is ruled by the joint structure, because the strength of the joints is decisive, whereas slope creep is ruled by the strength of the rock material. Investigations by Reitner et al. (1993) in a mountain built up by phyllonites dipping steeply to the north, have shown that slope creep dominates in the slope dipping to the north; whereas toppling dominates in the slope dipping to the south, because in the slope dipping to the south schistosity planes have an orientation optimal for toppling. In the slope dipping to the north; rock strength prevails, leading to slope creep, because the orientation of the schistosity planes does not make toppling possible.
Stresses in the toe area of toppling slopes are very high (Hittinger 1978), because the whole slope is lying on the toe. Thus the rock material fails and is very often completely crushed. Rock material strength is approaching its residual strength, which is the strength of the joints. So the complete mass is no longer discontinuous, which leads to slope creep in the toe region; whereas toppling occurs in the upper parts (Figure 10). This mechanism can be modelled very well by UDEC assuming the block material as a Bingham material (Poisel 1999).

**Slope Creep**
Slope creep is caused by the creep of rock masses, which is a material property (Langer 1979) and occurs in slopes as well as in foundations and around tunnels in rock. According to the decrease of the deviator stress with increasing depth below the slope surface, creep of the rock mass and therefore downslope displacements decrease continuously with increasing depth (up to 200 m). Typical features of a sagging slope are a tension crack (“Bergzerreissung”, Figure 11) in the upper slope surface and a bulging toe of the slope (Poisel 1998).

Figure 11: Tension crack (“Bergzerreissung”) in the upper slope surface of a creeping slope

Zischinsky (1966) investigated several cases of slope creep and derived a typical velocity distribution of such slopes (Figure 12). Zienkiewicz et al. (1975) showed that a slope of a Bingham material reveals continuously decreasing displacements with increasing depth. Thus
Slope creep can be effectively modelled numerically by a continuum code using a Bingham material. FLAC (Fast Lagrangian Analysis of Continua) from Itasca assigns, due to the time step algorithm routine, a behaviour like that of a Bingham material (Langer 1979) using a Mohr-Coulomb failure criterion.

Rock with little or zero cohesion deforms from the slope surface downwards (Figure 13), whereas, in a slope of rock with a higher cohesion, a block of rock very little deformation creeps downwards on a shear zone (Figure 14).

Figure 13: Continuously decreasing displacements in rocks with little or zero cohesion from the slope surface downwards (deformed FLAC mesh)

Figure 14: Very little deformed block creeping downslope on a shear zone in rock with higher cohesion (deformed FLAC mesh)

Zischinsky (1966) chose the term “sagging” (“Sackung”) for this type of failure mechanism. However, “sagging” (Absenkung) indicates a vertical movement (Heim 1932) while phenomena described by Zischinsky (1966) are triggered by displacements parallel to the slope surface. Hutchinson (1988) gave examples for “sagging” comprising extremely different mechanisms. Thus it seems better to avoid the terms “Sagging” and “Sackung” and to use “slope creep” and “Hangkriechen” instead.

3D-Effects
3D-effects have a strong influence on the stability of rock slopes, although they are very often neglected. Stability investigations by the shear strength reduction technique based on the definition of safety by Fellenius (1927) using FLAC3D have shown that a concave slope is much more stable than the straight slope, as space becomes narrower when the mass is
moving down. In contrast, the convex slope is slightly less stable than the straight slope (Figure 15; Preh 1999; Roth 1999; Zettler et al. 1999a; 1999b).

Figure 15: Numerical simulations of slope creep in a straight, a concave and a convex slope investigating 3D-effects using FLAC\textsuperscript{3D}

**Transition from Slope Creep to a Circular Failure**

Examples in the field show that slope creep may lead to a circular failure, due to high shear strains in the transition zone from rock remaining in place and displaced rock. Those high shear strains cause fracturing of the rock and decreasing rock strength in this zone, leading to localization of the zones failing in shear. This can be modelled effectively by FLAC (Cundall 1990). Thus, limit equilibrium methods for a circular failure and FLAC using the shear reduction technique give the same results (Zettler et al. 1999a; 1999b).

PFC can simulate large displacements. Therefore it is possible to model the transition from slope creep to a circular failure (rotational sliding) using PFC (Figure 16).

Figure 16(a): Numerical simulation of the transition from slope creep to a circular failure (rotational sliding) using PFC earlier stage: velocity vectors continuously decreasing with increasing depth; deformed, originally straight vertical lines show continuous deformation due to creep.
Figure 16(b): Numerical simulation of the transition from slope creep to a circular failure (rotational sliding) using PFC. Later stage: velocity vectors discontinuously decreasing with increasing depth; sheared through, originally straight vertical lines show discontinuous deformation due to rotational sliding

**Kink Band Slumping**

The term “kink band slumping” has been introduced by Kieffer (1998) describing a mechanism leading to S-shaped deformations of rock lamellae dipping steeper than the slope surface. Zischinsky (1966) and Nemcok et al. (1972) described similar slope deformations calling them “deep-seated creep” and “Sackung”. Numerical analyses (Preh 2004) using UDEC showed that this deformation is a consequence of rock creep and slipping of joints (Figure 17). As the upper parts of the moving rock mass slump and form normal faults (Figure 18) due to the slipping of joints, kink band slumping is not a special form of slope creep, which leads to tension in the upper parts of the slope and mostly to a tension crack.

Figure 17: UDEC model of kink band slumping
Figure 18: Normal faults in the upper part of a slope due to kink band slumping

WATER
Water is a very important factor, and it is possible to include the effect of water on the stability of a slope in a coupled mechanical-hydraulic analysis by the codes mentioned above. The real problem, however, is to obtain the necessary information. In most landslides, the hydraulic conditions are very complex and we never know them precisely enough in order to take them into account in an analysis which may be close to reality. In many cases, it is better to ignore groundwater and to take it into account by back calculating the angle of friction, which includes then the effect of water. However, this procedure is wrong for cases when differences in a reservoir level are big.

CONCLUDING REMARKS
The catalogue of landslide detachment mechanisms only comprises mechanisms having a clearly defined mechanical model. We often have to draw conclusions from a few vague surface structures as to what the interior structure or mechanism of a landslide detachment mechanism may be like. As in structural geology, it is an important criterion for the correct interpretation of structures to check if the mechanism in a certain case is possible not only in a geometrical or kinematical, but also mechanical way. Riedmüller (2003) pointed out that eventually only a mechanical model can identify the true causes of a landslide detachment mechanism. Moreover, the numerical models (especially for the initial failure mechanism and for the run out) and their results can only be as good as the models they are based on (e.g. topographic, geological, etc.).
The catalogue presented takes into account the geological setting and the geometry of the slope, the joint structure, the habitus of the rock blocks, as well as the mechanical behaviour of the rocks and of the rock mass (deformation and strength parameters). In order to classify and model a landslide detachment mechanism, close cooperation between geologist and engineer is therefore of paramount importance:

1) analysis of structures (observation and identification of discontinuities and fractures) by the geologist, because the geologist is qualified for this work,
2) synthesis of a mechanism by both the geologist and the engineer,
3) modelling by the engineer, because the engineer is qualified for this work,
4) interpretation of results by both the geologist and the engineer,
5) back to analysis of structures?

REFERENCES

Lausanne, 1421-1433.
Abstract: At the moment, we do not have an ideal model simulating both the initial failure mechanism of a rock slope and the run out. Thus we have to differentiate between rock slope failure models and models for the run out, based on the results of analyses of a particular rock slope failure. In general, a rock slope failure leads to the detachment of a rock mass consisting of a mass of blocks. Thus Punta Thurwieser Rock Avalanche and Frank Slide were modelled by the distinct element method, PFC\textsuperscript{3D}, modified for run out modelling. The simulations showed that the parameters necessary to get results coinciding with observations in nature are completely different, that the developments of mean particle velocities as well as of kinetic energy over time are completely different and that some 30 percent of total kinetic energy is rotational kinetic energy in Thurwieser, whereas the contribution of rotational kinetic energy in Frank Slide is zero. Thus the run out of the Frank Slide is a real “slide” of a coherent mass, whilst Punta Thurwieser run out is a rock mass fall with much internal movement. The parameters for a run out simulation therefore have to be chosen in such a way that the simulation gives a rock mass fall in one particular case and a slide of a coherent mass in another corresponding to the real conditions. Therefore, the prediction of the run out kinematics and the fixing of the parameters is a demanding task in each case when modelling run outs.

INTRODUCTION

The initial failure of a slope can consist of the failure of one or more discontinuities (slip, opening) and/or a failure of the rocks (plastic deformations, formation of new fractures). Slope movements may thus increase, leading to the formation of more new fractures and mostly to complete disintegration and loosening of the rock mass. Thus a rock slope failure leads to the detachment of a rock mass consisting of a mass of blocks (e.g. caused by block toppling) and a run out starts in the form of a rock avalanche (Voight & Pariseau 1978) where rock blocks interact during the phase from detachment to deposition.

Interpretations of observations of run out of rock slope failures (e.g. Heim 1932; Scheidegger 1973; Abele 1974), as well as experiments in the field and physical models, were the basis of run out prediction methods. Bagnold (1954) reported on experiments on a gravity-free dispersion of large solid spheres in a Newtonian fluid under shear, giving initial hints regarding the mechanisms of rock avalanches. Hsü’s (1975) experiments on bentonite suspensions suggested that the flow of thixotropic liquids is kinematically similar to the run out of rock slope failures. The results of the experiments showed that there is a positive semilogarithmic correlation between travel distance and the volume of the run out mass. Hungr & Morgenstern (1984) investigated the flow behaviour of dry sand at high velocities by laboratory flume experiments. Hutter & Savage (1988) performed experiments with a granular mass in a chute, with a bed made up of two plane portions joined smoothly by a curved transition. They found positive agreement between the model test results and a numerical model using a Lagrangian finite difference scheme. Thus many ideas on run out
prediction methods came from granular model tests.

Discontinuum (granular) mechanics methods model the run out mass as an assembly of particles moving down a surface. Each particle is followed exactly as it moves and interacts with the non-moving bedrock and with its neighbouring particles. Campbell & Brennen (1985) studied granular flows in a 2D numerical model using a code for molecular-dynamics calculations. Cao et al. (1996) studied the gravity driven granular flow of frictional particles down an inclined, bumpy chute by a numerical model which ensured the balance of momentum and energy.

Straub (1996) pointed out that a granular flow is a dissipative non-equilibrium system. It cannot be described without a fundamental modification to the classical thermodynamic framework. Such a modification is the assumption of a local equilibrium. He developed a model for rigid spheres with smooth or rough surfaces and for inelastic collisions. A function of restitution for the elastic properties and a Coulomb-type coefficient of friction for the surface roughness are the input parameters to model instantaneous dissipative interparticle collisions. Between the collisions, particles move along their ballistic trajectories.

Will & Konietzky (1998) used the Particle Flow Code, PFC$^{2D}$, by Itasca in order to analyze rock fall and rock avalanche problems. PFC models the movements and interactions of stressed assemblies of spherical particles being in or getting into contact with wall elements. Every particle is checked on its contacts with every other particle in every time step. Roth (2003) adapted the contact management in PFC$^{3D}$ in simulating rock avalanches in three dimensions.

Punta Thurwieser rock avalanche had a volume of some 2.2 million m$^3$ and reached a “Fahrböschung” of some 25°. Frank slide had a volume of some 36 million m$^3$ and reached a “Fahrböschung” of some 13°. Both run outs therefore fit well into the data of other mass movements showing that smaller volumes reach steeper “Fahrböschungen” and vice versa. It has to be assumed that this is due to the kinematics of run outs. Hungr (2007) reported that different parameters are necessary to model the run outs of smaller and larger volumes correctly using DAN (Hungr 1995). Therefore, Punta Thurwieser rock avalanche and Frank Slide were simulated using PFC$^{3D}$ in order to find out whether PFC$^{3D}$ can be used to simulate both steep and shallow “Fahrböschungen”, and which parameters are needed to model them.

**PFC$^{3D}$ AND ADAPTATIONS NECESSARY FOR RUN OUT MODELLING**

PFC models the movements and interactions of stressed assemblies of spherical particles either in or getting into contact with wall elements. The particles may be bonded together at their contact points to represent a solid that may fracture due to progressive bond breakage.

Every particle is checked on contact with every other particle at every time step. Thus, PFC can simulate not only failure mechanisms of rock slopes, but also the run out of a detached and fractured rock mass (Poisel & Roth 2004).

Rock mass falls can be modelled as an “All Ball model” and a “Ball Wall model”. An “All Ball model” (Figure 1(a)) simulates the slope as an assembly of balls bonded together. The simulation shows the failure mechanism of the slope due to gravity (Poisel & Preh 2004). After detachment of the moving mass, the run out is modelled automatically.
In the “Ball Wall model” (Figure 1(b)), the underlying bedrock is simulated by linear (2D) and planar (3D) wall elements (Roth 2003). Therefore, an estimate or a model of the failure mechanism of the slope and of the detachment mechanism is needed as an input parameter. However, in the “Ball Wall model”, the detached mass can be modelled using more and smaller balls with the same computational effort in order to approach reality better. The “Ball Wall model” offers the possibility to make use of the know-how related to run out relevant factors (e.g. coefficients of restitution, absorption, friction, etc.) applied in rock fall programmes (Hoek 1987; Spang & Rautenstrauch 1988).

![Figure 1: (a) All Ball model (Preh 2004), (b) Ball Wall model (Frühwirth 2004)](image)

According to observations in nature, several kinds of movements of the rock fall process (Broilli 1974) have to be distinguished during computation (Bozzolo 1987): free falling, bouncing, rolling and sliding.

In order to achieve an appropriate simulation of these different kinds of movements by PFC, some modifications have been necessary using the implemented programming language (Fish).

In order to model the free falling of blocks, neither the acceleration nor the velocity (ignoring the air resistance) is to be reduced during fall as a consequence of mechanical damping. PFC applies a local, non-viscous damping proportional to acceleration, to the movement of every single particle as a default. The local damping used in PFC is similar to that described in Cundall (1987). This damping model is the best suited for a quick calculation of equilibrium. There arises, however, the disadvantage of the movements of the particles being damped as well. Therefore, the local damping has been deactivated for all kinds of particle movements.

Elastic and plastic deformations occur in the contact zone during the impact of a block. Both the kinetic energy of the bouncing block and the rebound height are reduced by the deformation work.
The reduction of the velocity caused by the impact is modelled with the help of a viscous damping model integrated in PFC. The viscous damping model used in PFC introduces normal and shear dashpots at each contact (Figure 2). The relationship between the damping coefficient and the rebound height has been estimated by simulating drop tests (Figure 3).

The most important effect relevant to the run out is rolling resistance, because it is known that pure rolling of blocks in the model leads to more extensive run outs than observed in nature. The rolling resistance is caused by the deformation of the rolling body and/or the deformation of the ground and depends strongly on the ground and the block material (Preh & Poisel 2007).

Figure 2: Viscous damping activated at a contact with the linear contact model (Itasca 1999)

Figure 3: Relationship between restitution coefficient and critical damping ratio (Itasca 1999)
Due to these deformations, the distribution of contact stresses between the ground and the block is asymmetric. Replacing the contact stresses by equivalent static contact forces results in a normal force $N$, which is shifted forward by the distance of $c_{rr}$, and a friction force $F_{rr}$, opposing the direction of the movement (Figure 4).

The deceleration of the angular velocity caused by the rolling resistance is calculated using conservation of translational momentum (Eq. [1]) and angular momentum (Eq. [2]).

$$m \ddot{x}_s = -F_{rr}$$  \hspace{1cm} [1]

$$-I \cdot \dot{w}_{rr} = M_{rr}, \quad I_{sphere} = \frac{2}{5} \cdot m \cdot r^2$$  \hspace{1cm} [2]

where $M_{rr}$ is the resulting moment caused by the rolling resistance; $I$ is the principal moment of inertia and $\omega_{rr}$ is the angular deceleration. The kinematic link is established by the condition of pure rolling (Eq. [3]).

$$\dot{x}_s = w \cdot r$$  \hspace{1cm} [3]

The angular acceleration is defined by a finite difference relation in order to express the increment of the angular velocity per time increment. Therefore, the angular deceleration is calculated as

$$\Delta w_{rr} = \frac{-g \cdot c_{rr}}{r \cdot \left( \frac{r_{rr}}{r} - \frac{2}{5} \cdot r \right)} \cdot \Delta t$$  \hspace{1cm} [4]

The rolling resistance is implemented by adding the calculated increment of the angular velocity to the angular velocity calculated automatically by PFC at every time step (Eq. [5]).
1064

According to these considerations, the rolling resistance is an eccentricity, $c_{rr}$, or sag function, $u_{rr}$. The deeper the block sags, the greater the rolling resistance, $\Delta \omega_{rr}$. In classical mechanics, the rolling resistance is a function of the ratio of the eccentricity $c_{rr}$ to the radius $r$.

$$\mu_r = \frac{c_{rr}}{r}$$

This means that spherical blocks of different sizes have the same run out for the same rolling resistance coefficient. In nature, however, it can be observed that large blocks generally have a longer run out than smaller ones. Therefore, according to the damping model described, the run out is calibrated by the sag, $u_{rr}$.

Figure 5 shows that, within a deposit mass modelled by PFC, smaller particles rest at the bottom and larger particles at the top. This shows that the modified PFC routine can model inverse grading of run out masses observed in nature.

**Figure 5**: Inverse grading of deposit mass simulated by PFC

The run outs of Punta Thurwieser as well as of Frank Slide were simulated using the modified PFC routine described above. The input material was provided by the Organising Committee of “The 2007 International Forum on Landslide Disaster Management, Hong Kong - Landslide Runout Analysis Benchmarking Exercise” (2007). The digital terrain models (DTMs) of the terrain before (detached block of expanded rock) and after the mass movement were converted into triangulated meshes. These meshes were used to generate wall elements simulating the detachment and the terrain surface. The detached rock volume was modelled by particles (balls). The run out process was started by deleting the wall elements above the detached rock volume.

**PUNTA THURWIESER ROCK AVALANCHE**

The triangulated meshes converted from the digital terrain models (Organising Committee of “The 2007 International Forum on Landslide Disaster Management, Hong Kong - Landslide Runout Analysis Benchmarking Exercise” 2007) were used to generate 3,348 wall elements, simulating the detachment and the terrain surface. The detached, mostly dolomite rock was modelled by 2,632 particles (balls) with $r_{\text{min}} = 6$ m and $r_{\text{max}} = 11$ m (Figure 6). After starting the run out process by deleting the wall elements above the detached rock volume, 250,000 time steps were calculated.
According to the figures provided by the Organising Committee of “The 2007 International Forum on Landslide Disaster Management, Hong Kong - Landslide Runout Analysis Benchmarking Exercise” (2007) the terrain surface was divided into “glacier”, outcropping rock” and “glacial deposits” (Figure 6).

Table 1 shows the parameters necessary to get results coinciding with observations at Punta Thurwieser Rock Avalanche.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Detachment Area</th>
<th>Glacial deposits</th>
<th>Glacier</th>
<th>Outcropping Rock</th>
<th>Particle interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi$ [$^\circ$]</td>
<td>friction angle</td>
<td>40</td>
<td>60</td>
<td>15</td>
<td>45</td>
<td>60</td>
</tr>
<tr>
<td>$u_{rr}$ [m]</td>
<td>rolling resistance</td>
<td>0.5</td>
<td>0.4</td>
<td>0.6</td>
<td>0.4</td>
<td>-</td>
</tr>
<tr>
<td>$\beta_n$ [-]</td>
<td>critical damping ratio</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>(normal direction)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\beta_s$ [-]</td>
<td>critical damping ratio</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>(shear direction)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
The positions of the particles after 25,000 computation steps (Figure 7), after 50,000 computation steps and their final positions (Figure 8) show agreement with the documented rock avalanche spreading.

Figure 7: Particle positions after 63 seconds (25,000 steps)

Coinciding with reality, the numerical model showed that a small rock portion fell in the east. The run out distance of this portion in the model is much too large (long arrow in Figure 7). Presumably the roughness of the glacier in this area due to crevasses caused the blocks to stop at 3,100 m above sea level.
The final position of the particles and the deposition thicknesses in reality and in the model agrees fairly well (Figure 9). An even better coincidence between observed rock avalanche spreading and pathway of the particles in the numerical model could be achieved by a more exact terrain surface model (finer grid of the wall elements) and a more differentiated distribution of the glacial deposits and their parameters. This applies especially to the particles running out of the chute in the south-east of the outcropping rock (short arrow in Figure 7).

The development of mean particle velocity over time (Figure 10) as well as the development of kinetic energy over time (Figure 11) reveals that both reach a maximum after 30 seconds, just before the mass went over the break of the slope of the glacier (Figure 12). After 100 seconds, the maximum travel distance of the rock avalanche is reached and after that only internal movements occur before the mass comes to final rest (Figure 13).
Figure 9: Particle positions after stoppage and thickness of deposit in reality
Figure 10: Mean particle velocity (m/s) over time (s)

Figure 11: Rotational kinetic energy (red line), translational kinetic energy (black line), total kinetic energy (blue line) (J) over time (s)
Figure 12: Positions of particles after 30 seconds

Figure 13: Positions of particles after 100 seconds
FRANK SLIDE

The triangulated meshes converted from the digital terrain models (Organising Committee of “The 2007 International Forum on Landslide Disaster Management, Hong Kong - Landslide Runout Analysis Benchmarking Exercise” 2007) were used to generate 3,372 wall elements simulating the detachment and the terrain surface. The detached limestone rock was modelled by 19,691 particles (balls) with $r_{\text{min}} = 10$ m and $r_{\text{max}} = 15$ m (Figure 14). After starting the run out process by deleting the wall elements above the detached rock volume, 30,000 time steps were calculated.

Table 2 shows the parameters necessary to get results coinciding with observations at Frank Slide (Cruden & Krahn 1978).

Table 2: Best fit parameters for simulating Frank Slide

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Detachment Area</th>
<th>Surface</th>
<th>Particle interaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\phi$ [°]</td>
<td>friction angle</td>
<td>15</td>
<td>8</td>
<td>30</td>
</tr>
<tr>
<td>$u_{rr}$ [m]</td>
<td>rolling resistance</td>
<td>0.05</td>
<td>0.10</td>
<td>-</td>
</tr>
<tr>
<td>$\beta_n$ [-]</td>
<td>critical damping ratio (normal direction)</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
</tr>
<tr>
<td>$\beta_s$ [-]</td>
<td>critical damping ratio (shear direction)</td>
<td>0.5</td>
<td>0.5</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Figure 14: Frank Slide terrain surface modelled by 3,200 wall elements and detached rock volume modelled by 19,691 particles
The development of mean particle velocity over time (Figure 15) and of kinetic energy over time (Figure 16) reveals that both reach a maximum after 25 seconds. At this point of time, the centre of gravity of the mass is passing the river (Figure 17). The small amount of rotational energy reveals that a coherent mass is sliding down slope (Figure 16).

**Figure 15:** Mean particle velocity (m/s) over time (s)

**Figure 16:** Rotational kinetic energy (red line), translational kinetic energy (black line), total kinetic energy (blue line) (J) over time (s)
After 70 to 80 seconds, the mass comes more or less to rest (Figure 18). Afterwards, only minimal velocities and kinetic energies and therefore displacements (mostly backward movements) can be observed in the system (Figures 18 and 19). This corresponds well to observations in reality.
Figure 18: Particle positions after 65 seconds (15,000 steps)

The final positions of the particles in the PFC model (Figures 19 and 20) show no particles at rest in the floodplain. This corresponds well to the observation in reality that only minor damming of the Crowsnest River (indicating that the landslide eroded some material from the floodplain) was observed after the landslide.
The DTM of Frank Slide shows a ridge in the northern part (orographically left) of the final pathway. This ridge greatly influences the final distribution of the PFC particles. However, the path plan does not show this ridge. The differences between the slide deposit and the final distribution of the PFC particles may be caused by this ridge.
Due to the deposit thickness of 18 m on average in reality and to particle diameter distribution of 10 to 15 m, inverse grading of the deposit (Cruden & Hungr 1986) cannot be observed in the PFC model. In general, however, inverse grading of landslide deposits is the result of PFC modelling (see the section on PFC$^{3D}$ and Adaptations Necessary for Run Out Modelling). Due to computation time at present it is not possible to model more and smaller particles.

**COMPARISON AND CONCLUSIONS**
The comparison of the PFC models of Punta Thurwieser Rock Avalanche and of Frank Slide shows:

(1) The parameters necessary to get results coinciding with observations in nature (Tables 1 and 2) are completely different.

(2) The developments of mean particle velocities as well as of kinetic energy over time are completely different: there is much internal movement for a long time after reaching the
maximum run out distance in Punta Thurwieser, but there is just some backward movement down the opposite slope after run up in Frank Slide.

(3) Some 30 percent of total kinetic energy is rotational kinetic energy in Thurwieser, whereas the contribution of rotational kinetic energy in Frank Slide is zero.

Thus the run out of the Frank Slide is a real “slide” of a coherent mass (Cruden & Krahn 1978); whilst run out of Punta Thurwieser is a rock mass fall with much internal movement. The parameters for a run out simulation therefore have to be chosen in such a way that the simulation gives a rock mass fall in one particular case and a slide of a coherent mass in another corresponding to the real conditions. Hungr (2007) reported that different parameters are necessary to model the run outs of smaller and larger volumes correctly using DAN.

Punta Thurwieser rock avalanche as well as Frank slide fit well with the data of other mass movements showing that smaller volumes reach steeper “Fahrböschungen” and vice versa (Scheidegger 1973). However, it cannot be assumed that the volume is the only influencing parameter for the run out kinematics. Frank Slide and Randa Rock Fall had both approximately the same volume but very different “Fahrböschungen”. The detachment mechanism (sliding, toppling, etc.; Poisel & Preh 2004), the morphology of the detachment surface (more or less undulated in case of a sliding failure mechanism etc.) have significant influence on the degree of loosening of the moving mass and on the trigger mechanism of the run out. The morphology of the pathway of the run out also has a great influence on run out kinematics.

Therefore, the prediction of the run out kinematics and the fixing of the parameters is a demanding task in each case when modelling run outs.

REFERENCES
Dept. Civil Eng., University of Toronto, Toronto.
LANDSLIDE SIMULATION BY GEOTECHNICAL MODEL
ADOPTING A MODEL FOR VARIABLE APPARENT FRICTION COEFFICIENT

Fawu Wang
Research Centre on Landslides, Disaster Prevention Research Institute
Kyoto University, Japan

Kyoji Sassa
International Consortium on Landslides

Abstract: Sassa proposed a geotechnical model in 1988 to simulate the motion of landslides and founded the theoretical framework for landslide simulation. In 2002, based on the experimental results by ring-shear apparatus to simulate landslide motion, Wang & Sassa proposed a model for apparent friction coefficient changing, which is a key parameter in the geotechnical model. In the model for apparent friction coefficient changing, the landslide is considered as a two-layer structure, i.e. debris layer and sliding zone. During landsliding, the debris layer changes its thickness, while the sliding zone changes its shear resistance. In this paper, after a brief introduction to the two models, a parametric study is presented to show the relative importance of each parameter, and three case studies (1995 Fei Tsui Road landslide, 1995 Shum Wan landslide, and 1903 Frank slide) are presented to show the ability of the combination of the two models.

INTRODUCTION
As a natural phenomenon, landslide always causes tragic consequence, including untold numbers of deaths/injuries and huge economic losses. Especially, in recent years, with the residential and industrial developments expanding into unstable hillside area under the pressure of growing population, losses due to landslides are apparently growing. For prevention of the landslide disasters, prediction of landslide affecting area becomes more and more important.

Landslide motion prediction is considered to be a composite work. Its development is based on the understanding to the mechanism of landslide motion. Hungr (1995) classified the available simulation models for landslide motion to two categories: lumped mass models idealizing the motion of a slide as a single point (Perla et al. 1980; Hutchinson 1986), and models based on continuum mechanics (Sassa 1988). The friction of the sliding surface takes the most important role in both categories of models. The most famous lumped mass model, perhaps the first model to interpret the landslide motion is the “sled model”, which was originally proposed by Heim (1932). In this model, it is assumed that all energy loss during landslide motion is caused by friction.

Sassa (1985) put forward that the apparent friction angle is chiefly a combined result of the real internal friction angle, \( \phi_m \), and the pore pressure, \( u \), during landslide motion. He suggested that apparent friction angle, \( \phi_a \), should be approximately expressed by Eq. [1].
\[ \tan \phi_a = \frac{\sigma - u}{\sigma} \tan \phi_m \]  \hspace{1cm} [1]

where, \( \phi_a \) = apparent friction angle; \( \phi_m \) = internal friction angle during motion; \( \sigma \) = normal stress; \( u \) = pore pressure. The reason of “approximately” is that the cause of friction is not only friction, but others such as energy loss due to collision and viscosity may be somehow included.

Based on the concept of the apparent friction angle, (Sassa 1988) proposed a geotechnical model for the motion of landslides. It is a quasi-3D frictional model. Two motion equations were deduced from equilibrium of forces acting on a soil column, and a continuum equation was deduced from fluid dynamics. Through fixing the frame area for landslide motion, the motion of the sliding mass was estimated.

After the development of undrained ring-shear apparatus to simulate long distance motion of landslide in Disaster Prevention Research Institute, Kyoto University, some new findings were obtained. Based on the updated knowledge, Wang & Sassa (2002) proposed a model for apparent friction coefficient changing during landslide motion.

This paper will introduce an improvement on Sassa’s geotechnical model with adopting model for apparent friction coefficient changing, through a parametric study and three case studies.

**THE PRINCIPLES OF THE TWO MODELS**

The following are the principles of Sassa’s geotechnical model (1988) and Wang & Sassa’s (2002) model for apparent friction coefficient changing.

**Sassa’s Geotechnical Model for Landslide Motion (1988)**

Based on the kinematical principle of block motion and continuity principle of material during landslide motion process, (Sassa 1988) proposed the geotechnical model for landslide motion. Figure 1 illustrates a moving landslide and a column in it, and forces acting on the column.

The fundamental equations include two equations of motion (Eqs. [2] and [3]) in x and y directions respectively in a rectangular coordinate system, and an equation of continuity (Eq. [4]). Through finite difference calculating process, the distribution of the thickness of the sliding mass at each time step can be obtained for each mesh.

\[
\frac{\partial M}{\partial t} + \frac{\partial}{\partial x} (u_0 M) + \frac{\partial}{\partial y} (v_0 M) = g h \tan \alpha \frac{\partial h}{\partial x} - k g h \frac{\partial h}{\partial x} \left( \frac{u_0}{(q + 1)^{1/2} \sqrt{u_0^2 + v_0^2 + w_0^2}} \right)^{1/2} \left\{ h_t (q + 1) + h \tan \phi_a \right\} \]  \hspace{1cm} [2]

\[
\frac{\partial N}{\partial t} + \frac{\partial}{\partial x} (u_0 N) + \frac{\partial}{\partial y} (v_0 N) = g h \tan \beta \frac{\partial h}{\partial y} - k g h \frac{\partial h}{\partial y} \left( \frac{v_0}{(q + 1)^{1/2} \sqrt{u_0^2 + v_0^2 + w_0^2}} \right)^{1/2} \left\{ h_t (q + 1) + h \tan \phi_a \right\} \]  \hspace{1cm} [3]
\[ \frac{\partial h}{\partial t} + \frac{\partial M}{\partial x} + \frac{\partial N}{\partial y} = 0 \]  

where, $h$: thickness of the sliding mass; $M, N$: discharge in $x$ and $y$ direction per unit width, respectively ($M = u_v h, N = v_u h$); $k$: lateral earth pressure coefficient; $\tan \phi_h$: apparent friction coefficient of soil in the sliding zone; $h_c$: cohesion head (defined as cohesion $c = \rho g h_c$, it is zero after sliding for a long distance; $\rho$: density of the sliding mass); $\tan \alpha, \tan \beta$: inclinations of the intersection between the original slope surface and the $xz$ plane and $yz$ plane, respectively; $q = \tan^2 \alpha + \tan^2 \beta, \ w_b = -(u_v \tan \alpha + v_u \tan \beta)$.

In this model, the apparent friction angle $\phi_a$ in the moving area must be measured directly by the geotechnical test of sample taken from the site, or must be estimated from the friction angle and estimated pore water pressure using Eq. [1]. In (Sassa 1988), the value was estimated by the constant volume direct shear and the drained ring-shear test. After the undrained ring shear apparatus was developed (Sassa 1994), $\phi_a$ is obtained from Eq. [5].

\[ \tan \phi_a = \frac{\text{Measured shear strength at steady state} : \tau_{ss}}{\text{Initial total normal stress} : \sigma} \]  

Model for Apparent Friction Coefficient Changing during Landslide Motion

For the purpose to improve Sassa’s model as a tool for landslide motion prediction, a model for apparent friction coefficient changing during landslide motion was proposed by Wang & Sassa (2002). The following is the basic idea and the background for this model.

Figure 1: Geotechnical model for landslide motion (left) and forces acting on a sliding mass column (right)

It is assumed that the sliding mass composes of two layers, a relatively coherent slide debris layer moving on a thin sliding zone (Figure 2). This idea is similar to Hungr’s model. For the upper debris layer, the thickness will change during motion. It is a common phenomenon that the thickness of landslide mass becomes thinner during landslide motion, and finally, landslide stops and deposits. This means that the normal stress acting on the sliding zone will decrease, and then result in the corresponding increase of the apparent friction coefficient.
making an automatic stopping of landslide if the shear resistance is constant during landslide motion.

For the sliding zone, it can liquefy or not during the landslide motion, depending on its structure, saturated condition, grain crushing susceptibility and so on. A basic assumption is that it will reach the steady state after sliding for a certain distance. This phenomenon is well observed in our current researches by means of undrained ring-shear apparatus (Zhang & Sassa 1996; Sassa 1988; Okada et al. 2000). Zhang & Sassa (1996) found that, for the soil with the same initial void ratio under the undrained shearing for a long shearing distance, the shear resistances become to the same value, without any relationship to their initial normal stress. Figure 3 is a typical test result for this phenomenon obtained by Okada et al. (2000). Using sandy soils of Osaka formation from Hyogo Prefecture, Japan, shear velocity controlled undrained ring shear tests were conducted with different initial normal effective stresses. Finally, all of the stress paths go to the same steady state point, which means the shear resistance at the steady state was independent of initial normal stress. In some soils, the steady state value did not reach the same value. However, many cases had a similar trend more or less. We modeled the steady state shear resistance as an independent value of initial normal stress interpreting that there is a certain effective normal stress under which no further grain crushing will occur. Figure 4 is the model, and Eqs. [6] to [8] are the main equations.

Figure 3: Effective stress paths of four undrained ring-shear tests with different initial normal stresses on the Osaka formation sandy soils. The void ratios after consolidation for the four tests are the same as 0.71 (Okada et al. 2000).
Figure 4: Model for apparent friction coefficient changing during landslide motion
(Wang & Sassa 2002)

\[ \tau_a(h, B_a) = \tau_a + (\sigma(h) \tan \phi' - \tau_a)(1 - B_a) \]  \hspace{1cm} \text{[6]}

\[ \sigma(h) = \rho gh \cos \theta \]  \hspace{1cm} \text{[7]}

\[ \tan \phi_a = \frac{\tau_a(h, B_a)}{\sigma(h)} \]  \hspace{1cm} \text{[8]}

where, \( \tau_a \) is the shear resistance of the soil in the sliding zone at the steady state. \( \phi' \) is the effective residual friction angle. The two shear strength parameters can be measured in laboratory by ring-shear apparatus easily. \( B_a \) is the accumulation possibility of excess pore pressure. It is empirical index used to quantify the different condition of the traveling path during actual landsliding. It is mainly decided by the properties of the soils in the sliding zone, existing groundwater condition, drainage condition and so on. There are two extreme cases. When \( B_a = 0 \), it represents a completely dry condition; while when \( B_a = 1 \), it means a fully saturated and undrained condition. A suggested range of \( B_a \) value is presented in Table 1. \( \tau_a(h \cdot B_a) \) is the shear resistance of the sliding zone when the thickness of the sliding mass is \( h \) and the accumulation possibility of excess pore pressure at that location is \( B_a \). \( \sigma(h) \) is the total normal stress when the thickness of the sliding mass is \( h \). As the result, the apparent friction angle mobilized during motion \( \phi_a \) can be obtained by Eq. [8] and adopted in the motion Eqs. [2] and [3].

<table>
<thead>
<tr>
<th>Type and ( B_a )</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-type ( B_a = 0.0-0.1 )</td>
<td>Unsaturated sliding mass moving on unsaturated soil layer</td>
</tr>
<tr>
<td>B-type ( B_a = 0.9-1.0 )</td>
<td>Sliding mass moving on fully saturated soil layer or saturated sliding mass moving on impermeable layer (e.g. concrete channel)</td>
</tr>
<tr>
<td>C-type ( B_a = 0.1-0.9 )</td>
<td>Saturated sliding mass moving on unsaturated and permeable layer (unsaturated sandy layer)</td>
</tr>
</tbody>
</table>
Figure 5: Change of the apparent friction coefficient with thickness of sliding mass and $B_{ss}$

Figure 5 shows the variation of apparent friction coefficient with the thickness of sliding mass and $B_{ss}$. Three curves of different $B_{ss}$ representing the three types of landslides (A, B, C) cross at “D” point. From the left side to the right, the apparent friction coefficient increases with the decrease of the thickness. The apparent friction coefficient is smaller when the value of $B_{ss}$ is larger. While, when it goes beyond the “D” point to the right, the apparent friction coefficient becomes greater than $\tan \phi'$. It means that, when the upper debris layer becomes thinner than a critical thickness, $h_{cr}$ (defined by Eq. [9]), suction will take effect. In actual case, it is almost impossible to keep suction in the sliding zone because of the breaking of the sliding mass, so we assume that the apparent friction coefficient equals to $\tan \phi'$ when $h$ is smaller than $h_{cr}$.

$$h_{cr} = \frac{\tau_{ss}}{\gamma \cos^2 \theta \tan \phi'}$$  \hspace{1cm} [9]

Using the funding of IPL-M101 APERTIF Project-Areal Prediction of Earthquake and Rain-Induced Rapid and Long Traveling Flow Phenomena- by the Special Coordinating Fund for Promoting Science and Technology of the Government of Japan (Representative: K. Sassa) (Sassa 2004), the simulation programme was professionally improved. The program was written in C++ language. Its interface is very similar to MS-Word and is user-friendly. It can be easily operated in a Windows system. For the input data, it needs:

1. topographic data of slope surface and sliding surface. These data can be mesh data or control point data. If the input data are control point data, they will be transferred to mesh data before calculation by the program automatically.
2. mechanical parameters of sliding zone and sliding mass.

These parameters described in the above two models should be prepared. For the result of calculation, image files for arbitrary steps and video file can be output. The maximum speeds in $x$ and $y$ directions are shown in the image files of motion and the average speeds are given in an independent text file.
PARAMETRIC STUDY TO CONFIRM THE VALIDITY OF THE COMBINATION OF THE TWO MODELS

To confirm the validity of the combination of the two models, parametric study was conducted at first on a simple slope. Its dimensions are as follows. The width is 150 m, length is 210 m (90 m in slope part, and 120 m in plain part), and slope angle is 35° (Figure 6).

Figure 6: Dimension of the slope before landslide motion

The initial sliding mass was set as an ellipsoid. The axial dimensions of the ellipsoid are: 40 m for x direction, 75 m for y direction, and 50 m for z direction, while in y direction (slope direction), the axis is inclined 20 degrees downslope from horizontal. The epicenter of the ellipsoid is located at the surface of the slope, and lower half of the ellipsoid forms the sliding surface. Figure 6 shows the thickness distribution of the initial sliding mass. The volume of the sliding mass is 18,185 m³. Figure 7 shows a part of the simulating step results of the case when \( K = 0.3 \), \( \tau_{\text{ss}} = 20 \) kPa with the following parameters. \( B_{\text{ss}} = 1 \), effective friction coefficient for sliding zone is 0.577, unit weight of the sliding mass is 20 kN/m³. The maximum velocities in x and y directions are shown in the image figures with the step number and passing time. From Figure 7, the motion process of a landslide from acceleration to deceleration, and finally to stoppage state can be well observed.

It needs to be mentioned that the combination of the two models aims at the simulation after slope failure. So, at the beginning of calculation, an initial value of apparent friction coefficient which is low enough to make the first movement of the sliding mass happen is necessary. In this case, 0.6 was set as the initial value for the apparent friction coefficient. During the calculation, this value is only used in the first step. From the second step, the apparent friction coefficients are calculated with the model for apparent friction coefficient changing.

Figure 8 shows two series of results at the stoppage state to see the effects of shear resistance at steady state and earth pressure coefficient. Figure 8(a) is the 3D images; Figure 8(b) is central longitudinal section with \( \text{H/L} \) (Equivalent Coefficient of Friction); and Figure 8(c) is the planar distribution of the landslide mass.

From these results, the following tendencies can be observed, and these tendencies are reasonable from the viewpoint of soil mechanics.

1. Larger earth pressure coefficient will increase the lateral spreading tendency and make the sliding distance shorter. This will result in a larger equivalent coefficient of friction.
2. Larger shear resistance at steady state will decrease the sliding distance of the sliding
mass, and result in a larger equivalent coefficient of friction.

Figure 7: Time series result of simulation on a simple case with $K = 0.3$ and $\tau_s = 20$ kPa

<table>
<thead>
<tr>
<th>$K$</th>
<th>$\tau_s = 20$ kPa</th>
<th>$\tau_s = 30$ kPa</th>
<th>$\tau_s = 50$ kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>![Image]</td>
<td>![Image]</td>
<td>![Image]</td>
</tr>
<tr>
<td>0.6</td>
<td>![Image]</td>
<td>![Image]</td>
<td>![Image]</td>
</tr>
<tr>
<td>0.9</td>
<td>![Image]</td>
<td>![Image]</td>
<td>![Image]</td>
</tr>
</tbody>
</table>

(a) 3D image at the stoppage state

Figure 8: Parametric study results with the combination of the two models on an ellipsoidal slope
<table>
<thead>
<tr>
<th>(K)</th>
<th>(\tau_{ss} = 20) kPa</th>
<th>(\tau_{ss} = 30) kPa</th>
<th>(\tau_{ss} = 50) kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td>(H/L = 0.306)</td>
<td>(H/L = 0.344)</td>
<td>(H/L = 0.424)</td>
</tr>
<tr>
<td>0.6</td>
<td>(H/L = 0.315)</td>
<td>(H/L = 0.364)</td>
<td>(H/L = 0.466)</td>
</tr>
<tr>
<td>0.9</td>
<td>(H/L = 0.335)</td>
<td>(H/L = 0.384)</td>
<td>(H/L = 0.488)</td>
</tr>
</tbody>
</table>

(b) Longitudinal section at the stoppage state with the equivalent coefficient of friction \(H/L\)

<table>
<thead>
<tr>
<th>(K)</th>
<th>(\tau_{ss} = 20) kPa</th>
<th>(\tau_{ss} = 30) kPa</th>
<th>(\tau_{ss} = 50) kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.6</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.9</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(c) 2D image of the final deposition

Figure 8 (Con’t): Parametric study results with the combination of the two models on an ellipsoid slope
APPLICATION TO THREE CASE STUDIES
With the data supplied by the Review Committee for Benchmarking Exercise on Landslide Runout Analysis, the 2007 International Forum on Landslide Disaster Management, Hong Kong, we calculated three cases successful. They are: (1) 1995 Fei Tsui Road landslide, (2) 1995 Shum Wan landslide, and (3) 1903 Frank slide.

According to the request of the Review Committee for Benchmarking Exercise, the simulation results are reported with the following contents.

1) The mechanical parameters input.
2) Detailed plots of the flowing mass during motion and after stoppage. For comparison, the actual distribution area of the landslide is added in the last column following the calculation result.
3) Movement velocities at various steps.
4) Brief comments for each case study.

Our program cannot supply the movement vectors, because the finite difference method was used in the calculation. However, the motion can be observed well in the video file which will be presented in the forum.

Case Study on 1995 Fei Tsui Road Landslide
The following is a brief introduction to the Fei Tsui Road Landslide by Hong Kong Geotechnical Engineering Office (GEO). The 1995 Fei Tsui Road Landslide is a planar failure on a cut slope, with 14,000 m$^3$ debris sliding on a shallow-dipping, weak, kaolin-rich discontinuity within weathered volcanic rock, resulted in one fatality at Fei Tsui Road, Hong Kong. Details of the landslide mechanism, geological conditions, operating shear strength parameters and groundwater pressure of the failure, and deposition of landslide debris were established in a post-failure forensic investigation carried out by the GEO (2006a).

The landslide debris disintegrated during movement, and had a travel angle of about 24$^\circ$. The sliding surface day-lighted at the slope face of the cut slope at about 7 m above the ground fronting the slope, i.e. part of the debris jumped for a distance before hitting the ground. The digital elevation model has been built on the Geographic Information System platform based on the 1:1,000 topographic maps and spot heights. It is uniformly at 1 m-grid resolution.

Table 2 shows the mechanical parameters used in the simulation. Figure 9 shows the plots of the flowing mass during motion and after stoppage. Figure 10 shows the average velocities for different steps in x and y directions.

Comments: This is an easy case in our simulation. The velocity change in y direction shows an acceleration at first, and then deceleration. The main motion took about 20 seconds. Thereafter, it took about 30 seconds to become silent. All of the parameters used in the simulation (Table 2) are reasonable for soils.
Table 2: Mechanical parameters used in the simulation of the 1995 Fei Tsui Road landslide

<table>
<thead>
<tr>
<th>For sliding zone</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial apparent friction coefficient ($\tan \phi_{ia}$)</td>
<td>0.40</td>
</tr>
<tr>
<td>Accumulation possibility of excess pore pressure ($B_{sa}$)</td>
<td>0.90</td>
</tr>
<tr>
<td>Lateral earth pressure coefficient ($K$)</td>
<td>0.50</td>
</tr>
<tr>
<td>Effective friction coefficient at sliding zone ($\phi'$)</td>
<td>0.70</td>
</tr>
<tr>
<td>Shear resistance of sliding zone at steady state ($\tau_{ws}$)</td>
<td>30 kPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>For sliding mass</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of sliding mass</td>
<td>20 kN/m$^3$</td>
</tr>
<tr>
<td>Cohesion inside sliding mass</td>
<td>0 kPa</td>
</tr>
</tbody>
</table>

Water-blue line represents the source area of the landslide, and the red-purple line represents the motion boundary of the simulated landslide. The contour interval is 2 m for this case.

(GEO 2006a)

Figure 9: Plots of calculated landslide motion of the 1995 Fei Tsui Road landslide
Figure 10: Calculated movement velocities of the 1995 Fei Tsui Road landslide in x direction (U) and y direction (V)

**Case Study on 1995 Shum Wan Landslide**

The following is a brief description of the Shum Wan landslide by GEO. The open hillside failure, involving 26,000 m$^3$ ground materials, occurred on a heavily vegetated hillside after a prolonged heavy rainstorm. Details of the landslide mechanism, geological conditions, operating shear strength parameters and groundwater pressure of the failure, and deposition of landslide debris were established in a post-failure forensic investigation carried out by the Hong Kong Geotechnical Engineering Office and the findings are documented in GEO (2006b).

<table>
<thead>
<tr>
<th>Table 3: Mechanical parameters used in the simulation of the 1995 Shum Wan landslide</th>
</tr>
</thead>
<tbody>
<tr>
<td>For sliding zone</td>
</tr>
<tr>
<td>Initial apparent friction coefficient (tan $\phi_a$)</td>
</tr>
<tr>
<td>Accumulation possibility of excess pore pressure ($B_o$)</td>
</tr>
<tr>
<td>Lateral earth pressure coefficient ($K$)</td>
</tr>
<tr>
<td>Effective friction coefficient at sliding zone ($\phi'$)</td>
</tr>
<tr>
<td>Shear resistance of sliding zone at steady state ($\tau_o$)</td>
</tr>
<tr>
<td>For sliding mass</td>
</tr>
<tr>
<td>Unit weight of sliding mass</td>
</tr>
<tr>
<td>Cohesion inside sliding mass</td>
</tr>
</tbody>
</table>

The open hillside failure was fast-moving. It travelled down the densely vegetated hillside with negligible debris entrainment, across the two-lane road and severely damaged three shipyards and a factory, causing two fatalities and injuring five other people. The digital elevation model has been built on the Geographic Information System platform from the published 1:1,000 topographic maps and spot heights. It has a uniform at 5 m-grid resolution.

Table 3 shows the mechanical parameters used in the simulation. Figure 11 shows the plots of the flowing mass during motion and after stoppage. Figure 12 shows the average velocities for different steps in x and y directions.
Water-blue line represents the source area of the landslide, and the red-purple line represents the motion boundary of the simulated landslide. The contour interval is 2 m for this case.

Figure 11: Plots of calculated landslide motion of the 1995 Shum Wan landslide

Comments: This is also a simple case in our simulation. The mechanical parameters were applied for all of the calculation area. The acceleration-deceleration-stoppage process continuing for 27 seconds can be confirmed from the velocity change data. This case is a typical shallow landslide moving for long distance.
Fig. 12: Calculated movement velocities of the 1995 Shum Wan landslide in x direction (U) and y direction (V)

Case Study on Frank Slide
The following is a brief introduction of the Frank slide by Froese et al. (2007).

The Frank slide of 1903 was the deadliest landslide disaster of Canadian history, having destroyed a portion of the town of Frank with a loss of approximately 70 lives (McConnell & Brock 1904). It was a typical rock avalanche (for classification see Hungr et al. 2001), involving approximately 36 million m³ of fragmented limestone rock (Cruden & Krahn 1978). The rock detached from the ridge of Turtle Mountain over a front about 700 m wide, disintegrated into fragments, descended the 800 m high slope, crossed talus aprons and glacial drift benches and covered the floodplain of Crowsnest River and the opposite hillside in a deposit about 1.7 km wide almost 2 km long (Figure 13). The deposit is approximately 18 m thick on average. It exhibits well-developed inverse grading, with mean grain sizes ranging from sandy gravel at the base to boulders several metres in diameter at the surface (Cruden & Hungr 1986). The debris caused only minor damming of the Crowsnest River, indicating that the landslide eroded some material from the floodplain, but also that the maximum thickness of the deposit does not coincide with its proximal sector.

The geometric data is based on a “bare earth” Digital Elevation Model (DEM) obtained by a LiDAR survey with 1 m resolution in terms of elevation, provided by the Province of Alberta Geological Survey (AGS). The data was supplemented by additional survey points obtained from AGS. The model was then converted into a Golden Software “Surfer” grid files with 20 m grid spacing in both directions. The “Path Surface” model represents the elevation of the slide rupture surface and the slope of the mountain. Valley elevations have been adjusted approximately to simulate pre-slide conditions. Two grids represent the thickness distribution of the source area. One is the detached block of solid rock, the other is expanded in vertical thickness by 20% to account for fragmentation. A rectified orthophoto of the area at a resolution of 600 dots per inch was obtained and registered with the DEM.
Figure 13: The Frank Slide (photo courtesy of Prof. D.M. Cruden, University of Alberta)

<table>
<thead>
<tr>
<th>Table 4: Mechanical parameters used in the simulation on the Frank slide</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>For sliding zone</strong></td>
</tr>
<tr>
<td>Initial apparent friction coefficient (( \tan \phi_{ia} ))</td>
</tr>
<tr>
<td>Accumulation possibility of excess pore pressure (( B_s ))</td>
</tr>
<tr>
<td>For the area lower than 1,320m</td>
</tr>
<tr>
<td>For the area between 1,320m-1,600m</td>
</tr>
<tr>
<td>For the area higher than 1,600m</td>
</tr>
<tr>
<td>Lateral earth pressure coefficient (( K ))</td>
</tr>
<tr>
<td>For the area lower than 1,320m</td>
</tr>
<tr>
<td>For the area between 1,320m-1,600m</td>
</tr>
<tr>
<td>For the area higher than 1,600m</td>
</tr>
<tr>
<td>Effective friction coefficient at sliding zone (( \phi' ))</td>
</tr>
<tr>
<td>Shear resistance of sliding zone at steady state (( \tau_s ))</td>
</tr>
<tr>
<td>For the area lower than 1,320m*</td>
</tr>
<tr>
<td>For the area higher than 1,320m</td>
</tr>
<tr>
<td><strong>For sliding mass</strong></td>
</tr>
<tr>
<td>Unit weight of sliding mass</td>
</tr>
<tr>
<td>Cohesion inside sliding mass</td>
</tr>
</tbody>
</table>

* The shear resistance at steady state for the area lower than 1,320 m was measured by Sassa et al. (1998) with ring shear apparatus.

For the purpose of back-analysis of the 1903 slide, the DEM was modified to approximate the pre-1903 conditions, using evidence from the literature and the observation that the Crowsnest River did not change course significantly. The pre-slide slope surface in the source area was reconstructed approximately using historic photographs. The source volume was
adjusted so as to account for 20% “bulking” due to fragmentation during the landslide (fragmented volume is 36 million m$^3$). The DEMs were smoothed using the Surfer program, to improve numerical efficiency of models.

Unfortunately, information regarding movement velocity is not available. However, based on eyewitness interviews, McConnell & Brock (1904) conclude that the “time that elapsed between the first crash and complete rest did not exceed 100 seconds, and may have been somewhat less”.

In the map of the last figure of a map, the water-blue line shows the source area of the slide and deposition motion after sliding.

The thickness data which is expanded by 20% were used in this calculation. Table 4 shows the mechanical parameters used in the simulation. Figure 14 shows the plots of the flowing mass during motion and after stoppage. For comparison, the air photo and a map with boundary lines showing the source area of the slide and the deposition area after sliding were attached in the last two places. Figure 15 shows the average velocities for different steps in $x$ and $y$ directions.

Figure 14: Plots of calculated landslide motion of the Frank slide (water-blue line represents the source area of the landslide, and the red-purple line represents the motion boundary of the simulated landslide. The contour interval is 10 m for this case).
Figure 15: Calculated movement velocities of the Frank slide in x direction (U) and y direction (V)

Comments: It is very convenient to simulate the Frank slide with this program. However, because of the very large data size, we used 20 m×20 m as the mesh size instead of the 10 m ×10 m. As shown in Table 4, we used different parameters of accumulation possibility of excess pore pressure and lateral earth pressure coefficient for the area in different elevation. The reason is that the areas lower than 1,320m are nearby a river and the sliding surface was likely formed within the saturated layer, namely the landslide mass scraped the soil layer above the ground water table. For the shear resistance of sliding zone at steady state, a tested value of 17 kPa obtained by ring shear tests on the sample taken from this site by Sassa et al (1998) was used, while 80 kPa was assumed for the bedrock area. Comparing the distribution area of the sliding mass after stoppage with the actual distribution area of the Frank slide, the result is quite acceptable. The acceleration-deceleration-stoppage process can be observed from the velocity change of U and V in Figure 15, in both x and y directions, the velocity changes show reasonable distribution.

CONCLUDING REMARKS
Through the simulation on three different landslides and debris flows, the following conclusions are drawn:

(1) The combination of Sassa’s geotechnical model of landslide motion and Wang & Sassa’s model for apparent friction coefficient changing during landslide motion makes the simulation for landslide motion similar to the actual cases.

(2) The combination of the two models uses reasonable mechanical parameters in the simulation and can produce reasonable results. It means that this program can also be used for prediction of landslide motion, if we can obtain the mechanical parameters related to the landslide motion.

REFERENCES


REPORT ON BENCHMARKING EXERCISE OF LANDSLIDE DEBRIS RUNOUT AND MOBILITY MODELLING

Julian Kwan and H. W. Sun
Geotechnical Engineering Office, Civil Engineering and Development Department
Government of the Hong Kong Special Administrative Region

ORGANISATION
The majority of the modelling results were received from the participating teams by the end of October 2007, with supplementary information provided by some teams in November 2007. The Review Panel studied the results in October and November 2007, with assistance from the Hong Kong Support Team in collecting, analysing and summarising the results.

During the review, the Review Panel exchanged information and views via the Internet. Clarifications were sought from the participating teams as the need arose. By the end of November 2007, the modelling results were examined, and the key observations were made and agreed by the Panel. A draft review report was also prepared.

Professor O Hungr arrived at Hong Kong on 1 December 2007. During the following week, Professor O Hungr and Mr H N Wong went through the key issues relating to the review of the benchmarking results, finalised the programme of Day 3 of the Forum, and prepared on behalf of the Review Panel presentation materials that summarised the findings of the Benchmarking Exercise. Day 3 of the Forum was devoted to presentations, review and discussion of the modelling methodologies and results on the benchmarking cases.

PROGRAMME OF DAY 3 OF THE FORUM
The programme of Day 3 of the Forum on the Benchmarking Exercise is as follows:

<table>
<thead>
<tr>
<th>Time</th>
<th>Session</th>
<th>Activity</th>
<th>Presented by</th>
</tr>
</thead>
<tbody>
<tr>
<td>8:30 to 9:15</td>
<td>Overview (Chaired by H N Wong)</td>
<td>Overview of the Benchmarking Exercise</td>
<td>H N Wong</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Introduction of the Participating Teams</td>
<td>H W Sun</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Overview of the Modelling Methodologies</td>
<td>O Hungr</td>
</tr>
<tr>
<td>9:15 to 10:50</td>
<td>Presentation by Participating Team – Part 1 (Chaired by H N Wong)</td>
<td>Model: MADFLOW</td>
<td>J Chen</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: DAN3D</td>
<td>O Hungr</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: Pastor</td>
<td>M Pastor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: 3dDMM</td>
<td>J Kwan</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: RASH3D</td>
<td>M Pirulli</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: TITAN2D</td>
<td>M Sheridan</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Model: FLATMODEL</td>
<td>H W Sun</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(on behalf of M Hürlimann)</td>
</tr>
<tr>
<td>10:50 to 11:10</td>
<td>Discussion (Chaired by HN Wong)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
11:20 to 12:50
Presentation by Participating Team – Part 2 (Chaired by O Hungr)

- Model: Sassa-Wang
- Model: SHALTOP-2D
- Models: DAN3D & FLO-2D
- Model: Wang
- Model: PFC
- Model: TOCHNOG

Presented by
F Wang
A Lucas
J Cepeda
D Chan
R Poisel
G Crosta

12:50 to 13:10
Discussion (Chaired by O Hungr)

15:00 to 15:30
Review
Summary of Benchmarking Results
O Hungr & H N Wong

15:30 to 16:15
Discussion (Chaired by H N Wong)

The presentation materials are contained in a CD-ROM, which accompanies the Proceedings of this Forum.

KEY ISSUES DISCUSSED IN DAY 3 OF THE FORUM
Both the participating teams and the other participants of the Forum were actively engaged in discussions during the Forum. The discussions covered a range of issues, which included:

- modelling methodologies and solution approaches
- solution techniques
- rheological models
- assumptions made on internal stress and the effects on modelling results
- comparison of benchmarking results
- use of suitable rheological parameters in forward prediction
- choice of continuum and dis-continuum analysis
- need for 3-D modelling and provisions for split and merging of debris
- prospect for two- or multi-phase modelling
- shear rate and time-dependent effects
- effects of landslide initiation mechanism, consolidation and changes in terrain profile due to debris deposition/entrainment on debris runout path and mobility
- future research needs and directions
- areas for attention in real-life applications

A summary of the key points made by the participants during discussion is contained in Appendix A.

The effort made by the teams in this Benchmarking Exercise, as well as the value of conducting the exercise, was highly appreciated by the participants. The overall consistent outcome of the modelling results, despite that different solution approaches were adopted in the models developed/used by the teams, was found to be encouraging. Areas for future development were noted. The exercise served the intended purposes of taking stock of the progress made and degree of commonality of the methods developed, and facilitating
interaction among researchers and practitioners.

**DOCUMENTATION**
The Review Report prepared by the Review Panel is given in the Proceedings of the Forum. The participating teams have prepared a number of technical papers on their modelling methodologies and results, which are also published in the Proceedings.
APPENDIX A
SUMMARY OF KEY POINTS MADE BY PARTICIPANTS DURING DISCUSSION

HN Wong:
While many of the models give consistent results in the debris runout zone and distance in most of the benchmarked cases, there are some discrepancies in the assessed debris deposition profile and thickness. This may be related to the different assumptions made and methods adopted in dealing with the internal stresses.

Joanna Chen:
Mesh refinement helps to improve modelling the deposition profiles (an observation from modelling of deflected sand flow test). Unstructured nodes for modelling of the source area could be used instead of DEM in a regular grid, as for runout paths, to refine the simulation.

Oldrich Hungr:
For internal stress calculations, the Savage-Hutter formula will breakdown when slope angle is equal to friction angle. A modified S-H formula could be used to avoid this problem. Although it may not be theoretically ‘correct’, it will be practically realistic. The more diffused deposition at the distal end of the sand deposition in the deflected sand flow test could be due to the problem with the use of S-H formula. For back-analysis of Fei Tsui Road landslide, the use of uniform base friction $\phi$ appears to provide a better match with the deposition profile. If different $\phi$ values are used, it over-predicts the runout probably due to spreading, because for part of the debris, the internal friction value is equal to or less than the basal friction angle. When a uniform $\phi$ is used, the internal friction value is always greater than the base angle, so there is no problem in using of S-H formula.

Manuel Pastor:
The dam break problem is very good for calibration of codes based on shallow-water assumptions. Fei Tsui Road landslide is probably not very suitable for shallow-water assumptions. It would be better to model it using the differential approach, like TOCHNOG. The back analysis of Lo Wai debris flood is very difficult. The runout path is very sensitive to the topography and presence of building structures, which may not be fully represented by the DEM. For the change of debris characteristics along the runout path and at different time, the case should better be represented with an Evolution model.

Julian Kwan:
The S-H model is not strain-dependent (either at active or passive state), and there may be room for improvement here. This may be the reason of having an over-predicted spreading in the deflected sand flow case.

Marina Pirulli:
RASH3D has a range of features (e.g. mesh-dependent kinetic solver, various refined rheological models) to cater for different complex landslides. The results for Fei Tsui Road landslide, Frank slide and the deflected sand flow test are very consistent. For back analysis of the 2005 Tate’s Cairn landslide, the use of $\phi$ that equals to the runout angle produces the best match. The use of Voellmy model also produces similar results without predicting deposition along the runout path. However, if a quadratic model is used, it tends to predict deposition along the runout path and this has implications for the forward prediction.
Michael Sheridan:
TITAN2D is an open source code program and can be downloaded from www.gmfg.buffalo.edu. It adopts the grid-adaptive method and utilises parallel computing facilities. The code models the source material as piles (with no account for internal friction). We were able to have a 75% match for the Frank slide, where we over predict the spreading.

Julian Kwan:
Further details of the model used by the GEO will be published together with the results of more back analyses.

Manuel Pastor:
SPH code is more efficient than an Eulerian code, which requires a structured mesh and high computation effort.

Oldrich Hungr:
The SPH code is more efficient. However, SPH code has some numerical problems when the number of particles is small. For example, if we model channelised debris flow down a narrow gorge, the particles tend to line up in a row one-by-one.

Manuel Pastor:
Similar problem is observed if we have a small number of particles converging into one point.

Oldrich Hungr:
One of the great results of this Benchmarking Exercise is to see the consistency achieved by using different approaches, especially the Eulerian and Lagrangian approaches. It is almost unexpected. There are advantages and disadvantages of the two approaches but both are viable and it is very promising.

Manuel Pastor:
We made a lot of effort to convert our existing model into a SPH code. It is a difficult code to develop.

Michael Sheridan:
We are also converting our code into an SPH one. It is very difficult.

Oldrich Hungr:
Debris flow is complex and may have different characteristics at different parts of the runout path and at different times. Debris flow is multi-phase and heterogeneous, we have coarse particles at the front and fines at the back. The potential for further development of the particle codes is high. We can model different properties for different particles in different parts of the landslide.

Michael Sheridan / Oldrich Hungr:
We need more work and more data from real landslides to refine the particle models.

Marina Pirulli:
We work on Eulerian codes, e.g. RASH3D and FLO-2D. They require a lot of computation effort and we can only use fairly coarse grids or to reduce the analyses windows.
**Julian Kwan:**
When we developed our 3d-DMM model, several different methods were tested. SPH needs to use a large number of particles and small time steps. We have also been using FLO-2D. It is an Eulerian code and is very time consuming. The solutions obtained are generally consistent with field observations, although the simulation results are sensitive to the DEM of the topography. We have carried out FLO-2D analysis of the Lo Wai debris flood. In this case, we needed to modify the DEM to model obstruction to surface water flows by a fencing observed on site.

**Eric Leroi:**
As a user of the numerical model, I would like to know how do you obtain your field data? Have you been on the field to see the real case? Will you be able to improve your model if you have been on the field?

**Michael Sheridan:**
I am a field geologist. Field data, including detailed LiDAR data, and observations are useful and important for back analyses. If we were to do forward predictions, uncertainties in the terrain and parameters used should be considered. We can use a probabilistic approach but it is very time consuming. This is an area that we like to work on.

**Kyoji Sassa:**
All the cases match runout distance. In all these cases, runout distances are given and there is no need to study anymore. There are only small differences in debris distribution/deposition profiles. It does not help for forward predictions in practical applications.

**Oldrich Hungr:**
Our models need calibration so that we can perform predictions for events of similar characteristics. The model needs to fit a number of events of similar characteristics for us to gain confidence and reduce uncertainty. If the model cannot fit the case, the model needs to be changed but we cannot change the facts.

**Manuel Pastor:**
Back-analyses are useful for real landslide cases. In classical mechanics, results of controlled laboratory tests would help in gaining a better fundamental understanding. This is an area that I think we should pay our attention to.

**Joanna Chen:**
We use different parameters together with different rheological models. It would be useful if we can have a better appreciation and gain more confidence in the selection of suitable rheological model and parameters for different types of landslides. It would be useful if we could link the rheological model/parameters with some laboratory testing.

**Raymond Chan:**
The GEO needs to make slope safety decisions from time to time. Debris mobility modelling has provided us with a useful tool for hazard assessment, such as in dealing with the potential landslide risk in Tate’s Cairn. Rigorous scientific and technological advance would help us to make better decisions. I welcome further development in this area.
José Cepeda: 
Results of forward prediction for the Tate’s Cairn case are dependent on the DEM used. Smoothing the DEM would affect the velocity and peak discharge.

Dave Chan: 
Our model is based on conservation of energy and the results are sensitive to the assumptions made in side-forces, inter-slice force distribution and energy dissipation due to shear distortion. Other models may overestimate runout if they ignore internal stress distribution and the associated energy dissipation.

Rainer Poisel / Alexander Preh: 
In our particle flow code, completely different rheology and parameters are used. Kinematic energy involved in landslide is modelled but it is difficult to compare with other models.

Giovanni Crosta: 
TOCHNOG derives from an ‘open source’ finite element code but the runout modelling package is not part of the ‘open source’ code.

Manuel Pastor: 
I thought in SHALTOP-2D, you have the facility for reflection/damping. Have you done any calibration exercise on reflection/shock wave propagation?

Antoine Lucas: 
We don’t have this. It will be nice to have.

Rainer Poisel: 
For many landslide cases, it involve turbulence movement. For the Frank slide, it did not involve turbulence motion. So, the detachment mechanism could be important, e.g. toppling and sliding. The question is how can we know the mechanism in advance?

Giovanni Crosta: 
For the Frank slide, it was mainly sliding on a steep slip surface. I am not sure if it is very sensitive to the detachment mechanism as such. The mobility might be controlled by the topography, due to the presence of irregular ground surface profile. I am not sure if the initial detachment mechanism, say toppling, is important.

Oldrich Hungr: 
I am not sure if there is a relationship between the sizes of the particles in relation to the volume of landslide in the Particle Flow Code.

Alexander Preh: 
Yes, the number of particles is a controlling factor, especially if the landslide, as a whole, behaves as a continuum. For the Frank slide, it is not that sensitive. But I agree, we need to investigate further on the effect of size of particles.

HN Wong: 
Dave, your model takes account of energy changes associated with internal shear distortion during debris runout, which is not considered in the other models. How the shear strain of the debris and the internal energy dissipation in a dynamic situation be correctly modelled?
Dave Chan:
We are moving from momentum conservation to energy-based formulation. I found it is not right to adopt a momentum-based formulation. Once we adopted an energy-based approach, we can incorporate effects of shear distortion, turbulence, etc. through certain simplification and assumptions. We can use different constitutive models to consider internal strain.

Oldrich Hungr:
I strongly believe that momentum-based formulation, if it is correctly set up, should give us the same solution. Momentum-based or energy-based formulations are fundamentally the same. Some small differences can be expected if we adopt different assumptions for internal stress distribution.

Dave Chan:
The equations to be solved are highly indeterminate, it doesn’t matter if it is energy-based or momentum-based. We have to make assumptions, e.g. on the side-forces. The assumptions made should depend on the type of landslide we are analysing. The key is to incorporate the effect of shear distortion. We found that it is easier for us to adopt an energy-based approach to do so.

Oldrich Hungr:
Momentum-based method is easier to me. There are no obvious advantages or disadvantages for adopting either of the two approaches.

Andrew Leventhal:
I am amazed by your ability to simulate such complex events as the dynamics of landslides with simple models and parameters. As a potential user of your models, how do I assign parameters for my applications? Where can I find such guidance?

Oldrich Hungr:
We are in a state similar to that for slope stability calculations in the late 1960’s. We now have 40 years of experience in doing stability calculations and deducing parameters from investigations and laboratory tests. We don’t adopt laboratory test results directly in our slope stability models. We have to use judgement.

Rainer Poisel:
I am not sure if we are very good at slope stability calculations if we look at the friction angle of edge-to-edge contact between two blocks. Whatever you do, you have a hypothetical material and you have to find out the parameters.

Oldrich Hungr:
For the characterisation of the mass strength of rock, it is highly empirical and practical, which is derived from experience and back-analyses. If we can do it for rock, we should be able to do it for landslide mobility modelling.

HN Wong:
Back-analysis is a practical way of determining runout parameters for use in forward prediction. Given that reliable records of many historical landslide cases are available in Hong Kong, it is possible for us to systematically back-analysed the cases to establish the range of debris runout parameters and their probabilistic distribution. These can be applied to hazard and risk assessments.
Dave Chan:
We should first characterise the material and obtain the parameters. Nevertheless, this is an experience-based exercise, considering our current ability and limitations. The parameters have to be based on the type of rheological model used.

Giovanni Crosta:
Different models have different parameters/values for the same landslide. This may reflect our lack of understanding of the landslide mechanism. For example, we think the runout angle for larger landslide is smaller than that for small landslide. We do not have a good fundamental understanding. The analysis is also sensitive to many parameters, e.g. DEM, size of particles, as well as the geological model. On the other hand, I am not sure if the initial failure mechanism is very important for the movement mechanism after the first few seconds of the landslide.

José Cepeda:
We have studied volcanic debris flows in El Salvador based on some basic information such as grain size distribution and results of back-analyses of some Italian debris flows. We relied on a combination of field observations, laboratory tests, back-analyses of similar events to obtain the parameters and achieved very good results.

Alexander Preh:
The initiation mechanism has an influence on momentum and runout. For example, the initial rotation of blocks would have an influence on runout.

Dave Chan:
We have slope stability calculations and dynamic modelling for landslide mobility. Different parameters are used for the same material. The material changes from static status to dynamic status. Hopefully, we should be able to relate the material parameters in the two types of analyses, although it is unlikely that we will be doing combined stability and runout analyses in the near future.

Oldrich Hungr:
There are significant changes in the physics when the landslide mass moves and accelerates. We hope to be able to constrain the number of parameters needed. It is good that several groups are using the frictional rheology. There is one important consideration. In my opinion, frictional models need to incorporate rate dependent effects.

Su-Gon Lee:
You are dealing with debris flows. From a geotechnical point of view, most of the landslides occur along the boundary between soils/weathered rocks and rocks. The boundary is quite weak and you are modelling it as granular friction. I am not sure if it is right. The other consideration is the effect of rainfall (and the intensity). It is an important consideration, which influences the landslides.

Oldrich Hungr:
Yes, you are right. If the landslide involves dry material or occurs in dry weather, it will not be as mobile as one that is saturated.

HN Wong:
We should use parameters that are compatible with the characteristics of the landslide.
**Giovanni Crosta:**
I am not sure about rate dependency, especially in a rock avalanche. The material may behave differently at different stages. We may either model the average behaviour or simulate the changes with time.

**Oldrich Hungr:**
We model the average effect.

**Giovanni Crosta:**
We are far from having a complete model without having to use a lot of parameters.

**Oldrich Hungr:**
In some cases, especially for debris flow, we need to model rate effect in flow resistance.

**Manuel Pastor:**
In soil mechanics, most of the behaviour can be represented by elasto-plastic model, e.g. stress level, density. In dynamic cases with increasing rate of deformation, rate effect is to be dealt with by a viscous model. At a low strain rate, there is only one critical state. At a fast strain rate, there are effects on the critical state, e.g. affecting stress, volume, temperature and energy. I agree with Oldrich on rate effect.

**Antonio Gens:**
In soil mechanics, the critical state depends somewhat on strain rate. I am impressed by the capability of modelling landslide mobility. This is an impressive state-of-the-art. For the next step, we need to improve on the ability for forward prediction, based on field investigation results and laboratory tests. I feel, also, that we are still far from having all the fundamentals of the physics of landslide movement totally sorted out. I think, for instance, of the need of a coupled formulation including water flow and pore pressure dissipations. There are certainly opportunities for further research.

**Manuel Pastor:**
In geomechanics, we are to deal with pore pressure, segregation, etc. The process is complex and needs simplification for integration in numerical models. Pore pressure is important as we are working on integrating its effect in our model.

**Antonio Gens:**
You are doing very well indeed with your present theory. My basic points is that, I am still not sure if there is something basic that is missing and this is why more research is suggested.

**Dave Chan:**
Rate dependency should be addressed, but the question is how. There are variations between different parts of the landslide and between landslides. For some cases, this effect is more dominant than others. We need to look at this fundamentally.

**Oldrich Hungr:**
For some cases, we need to consider rate effect, but for some we do not need to.

**Suzanne Lacasse:**
The ability to undertake forward prediction is important. Have we come to a point that we have confidence in applying this in a real situation?
Oldrich Hungr:
It depends on the situation. If this is a small dry rockslide, the frictional model is reliable and the confidence is high. If wet material is involved, on entrainment or liquefaction is involved, there will be a range of parameters applicable and we have to provide the best estimate and range of estimates. It shows in this Benchmarking Exercise that we are able to benchmark runout distances using the adopted parameters to within a narrow band. We can do it within a narrow band of best estimate and bounds, although we cannot be exact.

Suzanne Lacasse:
Will you provide some summaries on this (the parameters used)? Will you give some tries?

HN Wong:
Yes, we will do so.

ML Lin:
If we simulate a debris flow, you may want to fill up local depression in your DEM before you run the analysis.

HN Wong:
As I have explained in my presentation, debris deposition can fill up some local ‘sinks’ in the DEM, debris barriers or deflection structures, and this may alter the debris flow path in some cases. In such circumstances, we need to manually amend the DEM.

Eric Leroi:
What do you need more, knowledge, data, benchmarking cases, parameters, working with decision-makers, etc.?

Johnny Cheuk:
Any laboratory tests to calibrate the parameters? I mean to calibrate intrinsic parameters from laboratory tests with parameters used in mobility modelling.

Oldrich Hungr:
In my opinion, ‘no’.

HN Wong:
Our current approach is semi-empirical. We cannot get the runout parameters from laboratory testing. I would recommend that we should get good case histories and back-analyse them, using rheological models that are representative of the types of events under consideration.

Dave Chan:
To be able to gain confidence, apart from back-analysing case histories we need to look back to flow mechanics and particle mechanics. We are not alone in this area, and others are working on it. We can refer to them and we may be able to incorporate some fundamental understanding in our work. We need to go back to basic and identify the key factors step by step. In this way, we can gain further confidence.

Lars Blikra:
So far, we are getting good and consistent calibration. Have you tried sensitivity analyses on DMM as for stability analyses, e.g. varying friction angle?
Oldrich Hungr:
I showed runout analyses of Frank slide using a matrix of two parameters and we vary them over a range and we zero them on the best prediction.

Pasquale Versace:
I am a hydrologist and I work on simulation of water cycles. The benchmarking results are too good and the real cases can be easily reproduced. That may be due to a large flexibility of the models and the capability of the models. The main issue is on the uncertainty involved in the modelling, especially in forward predictions.

Andrew Leventhal:
Acknowledging the usefulness and consistency achieved in the exercise, I have an observation to share, if I may. When we back analyse a (non-dynamic) soil slope, there is no unique answer – there are pairs of shear strength parameters, and variable pore pressure distributions. There are many ‘answers’ yet this is a situation that is, dare I say it, not as complex a problem as the debris flow scenario with which you are dealing. That is a more complex mechanism involving static and dynamic shear strength parameters for granular material, with pore water pressure and pore air pressure etc as well. We must remember that there is a heavy reliance on engineering judgement.

Luciano Picarelli:
Regarding the role of soil properties on the landslide behaviour we should clearly distinguish between our model, i.e. the way to reproduce what we ‘imagine’ may happen within the soil mass, and reality. To clarify what I mean, I would like to mention a lively discussion I had some years ago with a professor in hydraulics. He used a viscous model to simulate the run-out and velocity of flowslides in silty sands. These landslides often stop where the slope gradient is 5 to 10 degrees, a range of values which is very high for a viscous liquid. To me, the reason is very simple. It is due to dissipation of excess pore pressure which are induced at failure, enabling the increase in friction. In fact, through laboratory tests and flume tests we have demonstrated that flowslide triggering in volcanic ash in the Campania Region of Italy is a result of liquefaction. In contrast, my interlocutor had in his mind a viscous model and insisted that the landslide behaviour was related to increase in viscosity, so he artificially increases the viscosity in his model. Rate effect on clay has been investigated for quite some time (by Kenney, Vaughan etc.) and there are findings that there is no significant effect on the residual shear strength parameters, i.e. on $\phi'_r$. Morgenstern and Hungr also worked on rate effects of granular soils and found the same. So, if we think to rate effect as the impact of the deformation rate on soil properties, in my opinion this is limited, if there is any. The role of rate effects, instead, is prominent on the internal state of stress of the debris, causing for instance the triggering of excess pore pressures and favouring, or not, their dissipation.

Oldrich Hungr:
The previous tests done by me were on dry sands.

Luciano Picarelli:
I strongly believe that the rate effect is on building up of excess pore pressures, not on the shear strength parameters of the material. The results of very fast ring shear tests, e.g. those performed by Tika, show an increase or a decrease in the shear strength depending on material, and the most likely explanation for that is in the positive or negative excess pore pressure which is induced by shear. It is actually the result of interaction between the pore
fluid and the granular material.

**Manuel Pastor:**
I agree with you on the example about the hydraulics professor. We need to model dissipation of pore pressure. We have a model to couple consolidation effect and we used it for modelling debris flows in the Campania Region. We need to use a right model for the landslide although the same result can be achieved using different models. We need a logical model that considers the effects of changes in pore pressure. On the rate effect of granular material, there is an old paper in the Royal Society and it showed that the friction angle did not change much with velocity, but pore pressure was changing. You are right. We need to take this effect into account.

**Dave Chan:**
Back analysis is good for present use. There are two types of model that we see today. They are the continuum model and the dis-continuum model. I like the continuum model. There are a lot people using continuum models. We can incorporate effect of pore pressure and stress-strain in the continuum model. We have a lot of experience in using continuum models, including the parameters. Are we going along with continuum model instead of discrete model in future? I am happy to go along with continuum model.

**HN Wong:**
So far, we are largely dealing with a single-phase material in our runout modelling. How far are we away from using multi-phase models?

**Manuel Pastor:**
We are close to modelling this. We can deal with pore pressure effect. The problem is the effect of segregation. Even for the granular phase, we need to consider the variation of properties.

**Michael Sheridan:**
For TITAN, we have a two-phase model (water and solid). We still have a lot of problems to resolve. It is for the reason why we did not attempt any wet flow. We are not ready for this complex area and we are working on it and hope to publish our results next year.

**Oldrich Hungr:**
Two-phase model is more complex. We are calibrating single-phase problems with difficulties. For a two-phase model, we need to address, maybe, 10 times more variables, which are difficult to measure and calibrate. In foreseeable future (say for 100 years), if Dr Iverson is still alive, we may have a model that does not need calibration. The practical way in the next decade is to use a simple single-phase model and use suitable bulk parameters. The current simple and crude but flexible model is much better than having nothing. The current model is Okay for practical applications. The development of a two-phase model is for the future and for R&D.

**Rainer Poisel:**
Rock mechanics started as a field of continuum when we did not have adequate computing power. Now, we are all working on dis-continuum as it is able to better simulate the nature. Regarding runout, I would like to know if anyone here has a model to cover initiation of instability as well as runout? We have one, using dis-continuum modelling.
**Oldrich Hungr:**
We do not have one.

**Dave Chan:**
There is no doubt about the ability of dis-continuum modelling for geometric materials. I worked on dis-continuum modelling. It is not easy to characterise the micro-properties in a dis-continuum and the related properties of continuum that we are familiar with. The current way is an easy way out. This is why I now work on continuum models.

**FW Wang:**
I like to elaborate more about our model, as you said that our model results could not be used to compare with the others. In our cases, we have magnified the thickness by 5 times on the figures and this may be the reason for it.

**HN Wong:**
I meant that, as the details of the simulation results were not provided to us, it is difficult for your results to be compared with others.

**FW Wang:**
Anyway, I can provide more details. This is not a competition.

**Oldrich Hungr:**
Instead of having a two-phase model, we also change the r₀ in our single-phase model.

**HN Wong:**
This benchmarking exercise should only be the starting point for further interaction and closer collaboration among researchers and practitioners.
Closing Address
CLOSING ADDRESS

It is a great honour for me to deliver such a final speech, which should have been presented by Willy Lacerda as the Chairman of the JTC-1, who cannot make it to the conference and has asked me to take up this duty in his place.

I appreciate very much the subject of this Forum, which is not usual for scientists and researchers who prefer to discuss hazy things. However, the role of researchers can be of paramount importance at least as one of the men which are at the forefront. In fact, only a good knowledge of the mechanics of slopes can assist in learning what to do to avoid a landslide disaster. Furthermore, spreading of such a knowledge can be highly beneficial for others that have similar problems.

The structure of the Forum has been extremely well thought. A first session to share information about the situation in different countries which suffer from landslides; another one dedicated to the role of investigation to understand the mechanism of significant landslides around the world; one to provide information on innovative technologies which are being developed in the world; a last one to compare and assess the reliability of the tools that are currently used or could be soon adopted for analysis and risk assessment.

I am very satisfied about the content which emerged from the Forum. I have been updated about the experience in Hong Kong which remains a fundamental reference for many of us, but I have learnt very much from other countries which are moving towards a modern organization for risk governance and mitigation. Now I can eventually check the way we are running in Italy, especially after the catastrophic overflow of the Arno River in Florence in 1966, the earthquakes in 1976 and 1980, and the Val Pola rock avalanche in 1987: it is the right way. As a geotechnical engineer, I enjoy hearing very interesting landslide cases such as the one on the Aznalcóllar Dam and I have been shocked by other cases that I didn’t know in China, Malaysia and so on. Regarding the new technologies, I have been surprised by the fact that so many people, especially in Asia, are using optical fibres and TDR systems to monitor a number of different parameters and to check the quality of geotechnical works. These tools seem to me very promising for investigation. The wide use of InSAR and other systems adopted to watch is occurring on the earth is evident. Finally, the session today demonstrates that we can easily manage complex computer programs for landslide mobility just with a click.

Summing up, I think that this Forum represents an important step towards new advances in the capability to tackle catastrophic landslides. We should not lose such an experience and exploit this occasion for improvement and strengthening of our knowledge. Since the problem that we are dealing with is a prominent one for the safety and wealth of people living even tens of thousands of kilometers apart, further occasions must be found to share and compare experience, to improve expertise and search for new solutions. I think that a new workshop or conference should be dedicated to this subject in a couple of years. It should be open to people that are generally absent, as politicians and those who work at the forefront and have to take on themselves all the weight of critical situations. It should become a new occasion to share experience, spread information and study new solutions for complex problems. I hope that someone among us will think about that and propose a new meeting.
This has been a good occasion for me to meet again old friends and to establish new relations or strengthen old collaborations. Now I wish to thank all people engaged in the setting up of the meeting, the Organizing Committee, and especially the Chairman Ken Ho, the Advisory Committee, the staff of GEO and all other people who contributed to the success of the event, as the young women and the young men present in this room, who allowed the efficient running of all the sessions. Thank you very much once again.

L. Picarelli
Seconda Università di Napoli
Delegates of the 2007 Landslide Forum

Opening Address by R.K.S. Chan

Closing Address by L. Picarelli
## REGISTER OF PARTICIPANTS

<table>
<thead>
<tr>
<th>Name</th>
<th>Affiliation</th>
<th>E-mail</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blikra, Lars</td>
<td>Geological Survey of Norway, Norway</td>
<td><a href="mailto:lars.blikra@ngu.no">lars.blikra@ngu.no</a></td>
</tr>
<tr>
<td>Ho, Ping</td>
<td>Chongqing Construction Commission, China</td>
<td><a href="mailto:lhk66999@163.com">lhk66999@163.com</a></td>
</tr>
<tr>
<td>Canutti, Paolo</td>
<td>University of Firenze, Italy</td>
<td><a href="mailto:paolo.canutti@unifi.it">paolo.canutti@unifi.it</a></td>
</tr>
<tr>
<td>Ho, Ken</td>
<td>Geotechnical Engineering Office, Hong Kong</td>
<td><a href="mailto:kenho@cedd.gov.hk">kenho@cedd.gov.hk</a></td>
</tr>
<tr>
<td>Cepeda, José</td>
<td>International Centre for Geohazards, Norway</td>
<td><a href="mailto:jose.cepeda@geohazards.no">jose.cepeda@geohazards.no</a></td>
</tr>
<tr>
<td>Hutchinson, Jean</td>
<td>Queen's University, Canada</td>
<td><a href="mailto:jhutchin@geol.queensu.ca">jhutchin@geol.queensu.ca</a></td>
</tr>
<tr>
<td>Chan, Dave</td>
<td>The University of Alberta, Canada</td>
<td><a href="mailto:dave.chan@ualberta.ca">dave.chan@ualberta.ca</a></td>
</tr>
<tr>
<td>Jaafar, Kamal Bahrin</td>
<td>Public Works Department, Malaysia</td>
<td><a href="mailto:bahrin@jkr.gov.my">bahrin@jkr.gov.my</a></td>
</tr>
<tr>
<td>Chan, Raymond</td>
<td>Geotechnical Engineering Office, Hong Kong</td>
<td><a href="mailto:raymondkschan@cedd.gov.hk">raymondkschan@cedd.gov.hk</a></td>
</tr>
<tr>
<td>Jaboyedoff, Michel</td>
<td>University of Lausanne, Switzerland</td>
<td><a href="mailto:michel.jaboyedoff@unil.ch">michel.jaboyedoff@unil.ch</a></td>
</tr>
<tr>
<td>Chang, Ki-Tae</td>
<td>Kumoh National University of Technology, Korea</td>
<td><a href="mailto:ktchang@kumoh.ac.kr">ktchang@kumoh.ac.kr</a></td>
</tr>
<tr>
<td>Kim, Hak-Moon</td>
<td>Dankook University, Korea</td>
<td><a href="mailto:khm1028@dankook.ac.kr">khm1028@dankook.ac.kr</a></td>
</tr>
<tr>
<td>Chen, Joanna</td>
<td>Golder Associates, Canada</td>
<td><a href="mailto:joanna.chen@golder.com">joanna.chen@golder.com</a></td>
</tr>
<tr>
<td>Kim, Seung-Kyung</td>
<td>National Emergency Management Agency, Korea</td>
<td><a href="mailto:may1571@nema.go.kr">may1571@nema.go.kr</a></td>
</tr>
<tr>
<td>Cheuk, Johnny</td>
<td>The University of Hong Kong</td>
<td><a href="mailto:cyccheuk@hkucc.hku.hk">cyccheuk@hkucc.hku.hk</a></td>
</tr>
<tr>
<td>Corominas, Jordi</td>
<td>Technical University of Catalonia, Spain</td>
<td><a href="mailto:Jordi.Corominas@upc.edu">Jordi.Corominas@upc.edu</a></td>
</tr>
<tr>
<td>Koo, Y C</td>
<td>Fugro (Hong Kong) Ltd.</td>
<td><a href="mailto:yckoo@fugro.com.hk">yckoo@fugro.com.hk</a></td>
</tr>
<tr>
<td>Crosta, Giovanni</td>
<td>Univ. degli Studi di Milano, Italy</td>
<td><a href="mailto:giovanni.battista.crosta@unimib.it">giovanni.battista.crosta@unimib.it</a></td>
</tr>
<tr>
<td>Kwan, Julian</td>
<td>Geotechnical Engineering Office, Hong Kong</td>
<td><a href="mailto:juliankwan@cedd.gov.hk">juliankwan@cedd.gov.hk</a></td>
</tr>
<tr>
<td>De Silva, Suraj</td>
<td>Maunsell Geotechnical Services, Hong Kong</td>
<td><a href="mailto:suraj.desilva@maunsell.aecom.com">suraj.desilva@maunsell.aecom.com</a></td>
</tr>
<tr>
<td>Lacasse, Suzanne</td>
<td>Norwegian Geotechnical Institute, Norway</td>
<td><a href="mailto:suzanne.lacasse@ngi.no">suzanne.lacasse@ngi.no</a></td>
</tr>
<tr>
<td>Gens, Antonio</td>
<td>Technical University of Catalonia, Spain</td>
<td><a href="mailto:antonio.gens@upc.edu">antonio.gens@upc.edu</a></td>
</tr>
<tr>
<td>Lee, C F</td>
<td>The University of Hong Kong</td>
<td><a href="mailto:leecf@hkucc.hku.hk">leecf@hkucc.hku.hk</a></td>
</tr>
<tr>
<td>Lee, Su-Gon</td>
<td>University of Seoul, Korea</td>
<td><a href="mailto:sglee@uos.ac.kr">sglee@uos.ac.kr</a></td>
</tr>
</tbody>
</table>
Leroi, Eric  
URBATER, France  
E-mail: e.leroi@wanadoo.fr

Leventhal, Andrew  
GHD Geotechs, Australia  
E-mail: andrew.leventhal@ghd.com.au

Li, Eric  
Geotechnical Consulting Group (Asia) Ltd., HK  
E-mail: eric@gcgasia.com.hk

Li, Tie-Feng  
China Geological Survey, China  
E-mail: -

Li, Victor  
Victor Li & Associates Ltd., Hong Kong  
E-mail: vla@vla.hk

Lin, Meei-Ling  
National Taiwan University, Taiwan  
E-mail: linml@ntu.edu.tw

Lucas, Antoine  
Institute de Physique du Globe de Paris, France  
E-mail: lucas@ipgp.jussieu.fr

Lyttle, Peter  
US Geological Survey, USA  
E-mail: plyttle@usgs.gov

Mahmud, Mahadzer  
Kumpulan Ikram Sdn Bhd, Malaysia  
E-mail: mahadzer@ikram.com.my

Malone, Andrew  
The University of Hong Kong  
E-mail: awmalone@netvigator.com

McInnes, Robin  
Coastal and Geotechnical Services, UK  
E-mail: rgmcinnes@btinternet.com

Mohamad, Ashaari  
Public Works Department, Malaysia  
E-mail: drashaari@jkr.gov.my

Nadim, Farrokh  
Norwegian Geotechnical Institute, Norway  
E-mail: farrokh.nadim@ngi.no

Ng, Sam  
Geotechnical Engineering Office, Hong Kong  
E-mail: samuelkng@cedd.gov.hk

Omli, Ralph  
Norwegian Geotechnical Institute, Norway  
E-mail: rgo@ngi.no

Othman, Asbi  
Mohd Asbi & Associates, Malaysia  
E-mail: asbi@asbi-associates.com.my

Pappin, Jack  
Ove Arup & Partners, Hong Kong  
E-mail: jack.pappin@arup.com

Pastor, Manuel  
CEDEX, Spain  
E-mail: mpastor@cedex.es

Picarelli, Luciano  
Seconda Università di Napoli, Italy  
E-mail: picarell@unina.it

Pirulli, Marina  
Politecnico di Torino, Italy  
E-mail: marina.pirulli@polito.it

Poisel, Rainer  
Vienna University of Technology, Austria  
E-mail: rainer.poisel@tuwien.ac.at

Preh, Alexander  
Vienna University of Technology, Austria  
E-mail: alexander.preh@tuwien.ac.at

Pun, W K  
Geotechnical Engineering Office, Hong Kong  
E-mail: wkpun@cedd.gov.hk

Sassa, Kyoji  
Kyoto University, Japan  
E-mail: sassa@iclhq.org

Scavia, Claudio  
Politecnico di Torino, Italy  
E-mail: claudio.scavia@polito.it

Sheridan, Michael  
University of Buffalo, USA  
E-mail: mfs@geology.buffalo.edu
AUTHOR INDEX

A
Abdullah, C.H. 3
Alonso, E. 409

B
Bateman, A. 933
Blanc, T. 987
Bonnard, Ch. 79
Bouchut, F. 967
Bristeau, M.-O. 967
Bromhead, E.N. 343

C
Canuti, P. 587
Capparelli, G. 509
Casagli, N. 587
Castellanos Abella, E.A. 717
Cepeda, J. 813
Chan, D. 835
Chan, R.K.S. 1, 17
Chang, K.T. 579
Chen, J.H. 857
Chen, Z.Y. 375
Cheng, P.F.K. 667
Cheuk, J.C.Y. 391
Cheung, W.M. 667
Corominas, J. 49
Crosta, G.B. 875

D
d’Orsi, R.N. 71
Dalbey, K. 899
Damiano, E. 643
de Riso, R. 281
DiBiagio, E. 687
Diederichs, M. 599
Dremptetic, V. 987

F
Falorni, G. 587
Farina, P. 587
Fernández Merodo, J.A. 987

G
Galas, S. 899
Gens, A. 409
Gue, S.S. 3

H
Haddad, B. 987
Han, H.S. 579
Hancox, G.T. 429
Henchers, R. 125, 451
Herreros, I. 987
Ho, A.N.L. 579
Ho, K.K.S. 535
Hungr, O. 755, 919
Hürlimann, M. 933
Hutchinson, D.J. 599

I
Imposimato, S. 875

J
Jaboyedoff, M. 79

K
Kalenchuk, K. 599
Kjekstad, O. 261
Koc, O. 1059
Kumar, D. 899
Kvalstad, T.J. 465
Kwan, J.S.H. 571, 945, 1097

L
Lacasse, S. 99, 747
Lato, M. 599
Lau, J.W.C. 391
Lee, C.F. 857
Lee, S.G. 125, 451
Leoni, L. 587
Leroi, E. 169
Leventhal, A. 205
Lin, M.L. 235
Lo, D.O.K. 667
Lucas, A. 967
Lyttle, P.T. 251
M
Mahmud, M. 3, 485
Mak, S.H. 17
Malone, A.W. 387
Mangeney, A. 967
McDougall, S. 919
McInnes, R.G. 343
McKinnon, M. 919
Medina, V. 933
Mège, D. 967
Mira, P. 987
Mohamad, A. 3, 485
Morgenstern, N.R. 755, 835
Mothersille, D. 579

N
Nadim, F. 99, 465
Ng, K.C. 619

O
Olivares, L. 643
Omli, R.G. 687
Othman, M.A. 485

P
Palmieri, M. 281
Pastor, M. 987
Pastor, M.J. 987
Patel, D. 261
Patra, A. 899
Picarelli, L. 281, 509, 567, 643, 1111
Pirulli, M. 1015
Poisel, R. 1043, 1059
Preh, A. 1043, 1059
Pun, W.K. 667

R
Roddeman, D. 875

S
Sánchez, M. 987
Sassa, K. 319, 1079
Scavia, C. 1015
Sheridan, M. 899
Strout, J.M. 687
Su, M.B. 555
Sulaiman, M.J. 485
Sun, H.W. 391, 571, 945, 1097

T
Tran, D. 835

V
van Westen, C.J. 717
Versace, P. 281, 509, 643

W
Wang, F.W. 1079
Wang, J.F. 579
Wang, S.J. 375
Wang, X.B. 835
Winter, M.G. 343
Wong, H.N. 535, 619, 755
Wu, H.L. 235, 555

Y
Yin, Y.P. 375
Yusof, M.A.M. 3