SIMSG ISSMGE

ISSMGE Bulletin

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International Society for Soil Mechanics and Geotechnical Engineering

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Inside This Issue

Editorial Board

Message to ISSMGE Members from Gabriel Auvinet, Vice-President for North America

As Vice-President for North America, I am deeply honored to address this message to all members of ISSMGE.

I would like to share with you some reflections on the present state of Geotechnical Engineering in North America and on the new trends I can perceive as the 75th anniversary of ISSMGE is celebrated.

The North American region includes only three member countries: Canada, USA and Mexico, a small number when compared to other regions such as South America, Asia and Europe. The individual membership in the ISSMGE represents however close to 20% of the grand total membership (approximately 19,000) of all member societies around the globe.

The three member societies of the region are extremely active and have a strong presence and influence in the engineering community and in the society in general in their respective country as well as internationally. It would be too lengthy to expose detailed information regarding the activities of each of the three member societies in this message. Data will be found on their excellent individual web sites:

Canadian Geotechnical Society (CGS, Canada): www.cgs.ca
Geo-Institute (G-I, ASCE, USA): www.geoinstitute.org
Sociedad Mexicana de Ingeniería Geotécnica (SMIG, Mexico): www.smig.org.mx

Message to ISSMGE Members (continued)
Gabriel Auvinet

The Canadian Geotechnical Society (Société Canadienne de Géotechnique) organizes each year a highly professional national meeting (GEO HALIFAX 2009, GEO2010 Calgary, Toronto 2011 Pan-American /CGS Conference) but also a number of special national or international conferences and short courses. National meetings provide a good occasion to confer prestigious awards upon distinguished members. The Society's main written (hard copy) communication medium is a newsletter called "CGS News" which appears in the quarterly journal "Geotechnical News" published by BiTech from Vancouver, British Columbia, Canada. The CGS News features short articles on what is happening within the CGS and other news items of specific interest to the members of CGS. Although not connected to the famed *Canadian Geotechnical Journal*, the Society has remained in close contact with the Journal and is a great supporter of it. The present president of CGS is Bryan Watts.

The Geo-Institute of the American Society of Civil Engineers is the US National Member Society for ISSMGE. G-I national meetings are among the largest technical events of this type organized in the world, with attendance of the order of 2000 participants (GeoFlorida, 2010; GeoFrontiers, 2011). Quality is associated with quantity in these meetings as well as in the numerous special conferences and courses organized or endorsed by the Geo-Institute. The Geotechnical Engineering journals published in the US, including the *ASCE Journal of Geotechnical and Geoenvironmental Engineering* and the *International Journal of Geomechanics*, are among the most prestigious in our profession. The Geo-Institute actively encourages development of continuing education short courses and webinars to disseminate technical information among its members.

ISSMGE membership in North America

Message to ISSMGE Members (continued)
Gabriel Auvinet

Philip King (right) is the new president of the Geo-Institute, ASCE; Larry Jedele will serve on the Board through 2012 as the past president.

The Mexican Society of Geotechnical Engineering (SMIG, formerly Mexican Society of Soil Mechanics) is the smallest but not the least active of the three North-American Societies. National meetings are held biennially (Acapulco, 2010) and a large number of lectures, including the prestigious *Nabor Carrillo lecture*, special conferences and short courses are organized to deal with topics as different as “Laboratory testing” and “Freud for engineers”. A new journal “*Geotecnia*” is now published by SMIG. A well-illustrated commemorative volume untitled “*El Siglo de la Mecánica de Suelos (Soil Mechanics’ Century)*” was published on the occasion of the fiftieth anniversary of the society. A large collection of special technical publications and conference proceedings is also available from SMIG. Recently, stress has been put on improving communication between members through the web and social networks. The present SMIG president is Juan de Dios Alemán.

The three member societies of North America are very active in ISSMGE Technical Committees and in particular in those hosted by the region: TC 102 “Ground Property Characterization from In-situ Tests” (USA), TC 206 “Interactive Geotechnical design” (Canada), TC 208 “Stability of Natural Slopes” (Canada), TC 209 “Offshore Geotechnics” (USA) and TC 214 “Foundation Engineering for Difficult Soft Soil Conditions” (Mexico)

International relations between the member countries of the North American Region but also between these countries and the rest of the world and in particular with South America are being actively developed. An agreement of cooperation was signed between G-I (USA) and SMIG (Mexico) on October 7th, 2009 in Alexandria, Egypt.

Message to ISSMGE Members (continued)
Gabriel Auvinet

Signature of a collaboration agreement between Geo-Institute, ASCE and SMIG (Alexandria, Egypt, 2009)

The Pan-American Conference on Soil Mechanics and Geotechnical Engineering, organized every four years since 1959, is still the main occasion to foster exchanges between North and South America and to promote the development of soil mechanics and geotechnical engineering in some countries where these topics have not received adequate diffusion. It originally surged from the fact that many of the best specialists in Latin-America were former students of US Universities such as Harvard, Illinois, MIT, Berkeley, etc. An additional interest was also expressed by many consulting or equipment companies of North America wishing to develop their contacts and activities in the rest of the continent. These interests are still very much alive now taking into account the large number of specialists of Latin origin working in North America (and to some point the other way around). The weight of the different nations in the world concert is evolving rapidly with the surge of new powerful emerging countries but the interest of the Pan-American Conference remains intact.

Recently (October 2-6, 2011), the 14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering was held in Toronto, Ontario, Canada, together with the 64th CGS National Conference and the 5th Pan-American Conference on Teaching and Learning of Geotechnical Engineering. The 2011 Pan-Am/CGS Geotechnical Conference also featured a comprehensive Trade Exhibition where suppliers to the geotechnical and hydrogeological industry were able to showcase their latest products and services. The conference technical program of this very successful event enhanced opportunities for interaction between academics, practitioners, designers, contractors and owners from North, Central and South America. The prestigious Casagrande Lecture was brilliantly delivered by Dr. Kerry Rowe (Queen's University). To promote a wide participation in this event of member societies of the North and South America regions, a special previous meeting of the Pan-American Committee was organized in Gramado, Brazil (during COBRAMSEG2010, August 17-22) with participation of delegates from 15 member countries. This committee met again during the Pan-American Conference on October 4th 2011 in Toronto. An updated agreement between the member countries of the two regions was approved and the venue of the next Pan-American Conference (2015) was unanimously assigned to Buenos Aires, Argentina.

Message to ISSMGE Members (continued)
Gabriel Auvinet

A touristic/technical visit to the Niagara Falls was organized during the Pan-Am /CGS Conference in Toronto

The 75th anniversary of ISSMGE is an important landmark. ISSMGE is now a respectable 75 years old lady, with the magic power of renewing herself constantly thanks to the inflow of new young members and to the reluctant fading away of old warriors. Homage to the pioneers of the past is an important source of inspiration for the young members. On January 20th 2011, SMIG organized in Mexico City a Special Symposium to honor the memory of the late Prof. Leonardo Zeevaert, with participation of Jean-Louis Briaud, and William Van Impe, respectively President and former President of ISSMGE.

An anniversary is always a good time for reflection. It is an appropriate occasion to look back to the past but also to assess the present in order to prepare the future. The past, present and future of Soil Mechanics and Geotechnical Engineering was the main topic of a special luncheon session celebrated in Toronto on October 3rd 2011. The comments on the past of ISSMGE by Prof. Norbert Morgenstern provided a deep insight into the different stages of the evolution of our International Society. Two young members presented their vision of what can be expected from the future. As far as the present is concerned, I was glad to comment on some of the many reasons why we should feel satisfied with the present state of our profession in North America.

Message to ISSMGE Members (continued)
Gabriel Auvinet

A strong dynamism is observed in the activities of our members. Challenging topics are being dealt with including: Geotechnical data management, Geostatistics, Variability and uncertainty, Physical and numerical modeling, Reliability and risk analysis, Ground improvement, Sustainability, Energy piles, Geosynthetics, Geohazards, Geoenvironmental engineering, Land subsidence, Offshore engineering, New concepts in deep foundations and Underground structures. A large number of topics could be added to the above list. Some of them are still vying to be accepted as significant contributions to geotechnical practice. This is the case of some sophisticated approach such as micromechanics studies on soils or soft computing applications. The importance of basic research on this kind of topics should however be recognized since future progress may depend on them. Far from being stalled, our profession is actively pursuing new goals to better satisfy society's demands.

The large attendance of members participating in national, international and special conferences is also impressive. A special motive of satisfaction is the increasing number of special events organized for and by young members.

First symposium of young geotechnical engineers, Mexico City, February 19, 2010

New advances in computer science have a strong impact on the nature and intensity of our activities. Access to technical information is becoming unlimited and instantaneous; a new collective brain and memory is being continuously created. Communication between members of the same society and of different societies is improving at an impressive rate, and this is just a start since new systems such as our new "*Geoworld*" have just been implemented recently. Access to the best lecturers in our profession will become easily available in the future through *webinars*. A better diffusion is also given to the excellent Geotechnical Engineering journals published in the region.

Soil Mechanics and Geotechnical Engineering in North America is a buoyant and many-faceted specialty. Its brilliant and creative activities in the present are a guarantee of a promising future.

ISSMGE President progress Reports
Professor J-L. Briaud

730 days progress report

Distinguished Colleagues, Dear Friends,

This is my twenty fourth progress report after 730 days as your President. Note that previous reports are on the ISSMGE web site (http://www.issmge.org/) under "From the President" if you need them. It has been 2 years since you elected me. You may recall that on these anniversary reports I take the liberty to talk about things that are not related to geotechnical engineering but more related to life in general. But before I do that I will make an exception to help my friend Professor Antonio Correia, Chair of the ISSMGE TC on Transportation Geotechnics who will offer the second ISSMGE webinar on Intelligent Compaction on 25 Oct. 2011. I recommend it highly and will be a participant myself. If you wish to attend this webinar, please contact Hanna, my assistant, at hprichard@civil.tamu.edu.

In this report, I would like to share with you some of the most beautiful music I have come across in my life, music which has lifted me up in difficult times, music which has made me appreciate the beauty of life, music which makes me vibrate and makes me feel so alive. My parents forced me to play the piano until the end of high school and I hated every minute of it. Yet when I arrived in university I started to play by myself and music became a faithful companion. I ended up playing piano jazz in bars in Paris to earn some money as a student! and now I am so thankful that my parents forced me. Yes, I must admit it, my parents knew better! So here are some links that I have selected. There is classical music, jazz music, folksy music, and others. This selection is by no means all encompassing and many very classic pieces are not listed; nevertheless, it is a snap shot of my music world. Some of these links are preceded by commercials which I could not avoid; you can skip through them and get to the song.

http://www.youtube.com/watch?v=qKoX01170l0. The Song of the Birds by Pablo Casals invited by Kennedy at the White House in 1961. Simple beauty. When I hear this, I keep thinking how lucky we are to live among people who can generate such extraordinarily simple beauty and I am overwhelmed.

http://www.youtube.com/watch?v=-3WWZQyPs30&feature=related. Schubert Impromptu Op. 90 No. 2. Deliciously light and gliding. I get goose bumps when I hear this one.

http://www.youtube.com/watch?v=Nx7vOb7GNBg Asturias from Isaac Albeniz. Ana Vidovic. Masterful piece which surely wakes you up.

http://www.youtube.com/watch?v=9CDoJFmdFgA&feature=related. Minor swing by Jango Reinhardt. The master Django Reinhardt with Stephane Grapelli. This jazz piece is so uplifting and happy. When I hear this piece of music, it lifts up my spirit regardless of my mood. This is the music I chose for the opening dance with my daughter Natalie at her marriage in 2002!

http://www.youtube.com/watch?v=B6uXGSTfz_4&feature=related. The same piece of music but by virtuosos, epoustouflant!

http://www.youtube.com/watch?v=kISZ3vGJ2AA Jimmy Shand. How can you not be happy when you hear this one. It sure gives me plenty of energy on my way to work in the morning. I prefer that to coffee!

http://www.youtube.com/watch?v=Q8Tiz6INF7I&feature=related. Hit the road jack, Ray Charles. So much talent, so much life, so much fun.

http://www.youtube.com/watch?v=3PkXpCsgj5U&feature=related. Russian Red Army Kalinka. What a beautiful and classic song. Strength, passion, and nuances. Our own voice is such a wonderful instrument.

ISSMGE President progress Reports (Continued)
Professor J-L. Briaud

http://www.dailymotion.com/video/x28dnn_louis-armstrong-wonderful-world_music#rel-page-2. What a Wonderful Word by Louis Armstrong. Indeed what a wonderful world!

http://www.youtube.com/watch?v=XO4IA-1ioJQ Markahuasi los refranes. There is something special about the flute, you sense the breath of the player and communicate even better.

http://www.youtube.com/watch?v=44sp4W3WiBk&feature=rellist&playnext=1&list=PL643BBBF2A926310C. Antonio Carlos Jobim – Desafinado. I guess if you feel too excited about something. This may be the one to listen to. What calming and charming at the same time. The rhythm is unparalleled.

http://www.youtube.com/watch?v=1TD_pSeNelU Willy Nelson, On the road again. I play this one in the car every time I drive to the Houston Airport to catch a plane and go visit my friends geotechnical engineers throughout the world.

I just noticed that in preparing this report I listed as my favorite music, music from many different countries. Indeed music has no boundaries, no need for translation, everyone understands and appreciates. What a beautiful common language that we can share at any time. Have a great day and do share some of your favorite music with me if you wish.

Please relay this report to your membership.
Best wishes,
Jean-Louis BRIAUD
President of ISSMGE
International Society for Soil Mechanics and Geotechnical Engineering

CONFERENCE NEWS

IS-Seoul 2011; Fifth International Symposium on Deformation Characteristics of Geomaterials

This conference took place from September 1st to 3rd, 2011, in Seoul, Korea under the auspices of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) TC 101 and the Korean Geotechnical Society (KGS). This conference focused mainly on the recent advances in laboratory testing technology, applications of advanced laboratory as well as testing to filed problems, and values of technical developments in practice. This conference attracted 182 paper submissions (including 17 papers published in a special edition of the *Soils and Foundations* journal) and almost 320 participants from 31 countries.

Among many oral presentations, participants were able to enjoy eminent invited lectures as listed below;

The 1st Bishop lecture, sponsored by ISSMGE TC101:

Prof. Fumio Tatsuoka on *Laboratory stress-strain tests for developments in geotechnical engineering research and practice,*

Symposium Keynote Lectures:

Prof. Federica Cotecchia on *Investigating the influence of microstructure, loading history and fissuring on the clay response,*

Prof. Antonia Viana da Fonseca on *the Interpretation of conventional and non-conventional laboratory tests for challenging geotechnical problems,*

Prof. Kenneth H. Stokoe II on *Field evaluation of the effects of stress state, strain amplitude and pore pressure generation on shear moduli of geotechnical and MSW materials,*

Prof. Satoru Shibuya on *Case study on rainfall-induced behavior of unsaturated soils in natural slopes and reinforced-earth walls,*

Prof. Malcolm Bolton on *Using centrifuge models to define deformation mechanisms and generate design methods,* and

Prof. Richard Finno on *Identification of constitutive parameter with field performance data.*

Out of many submitted papers, 17 papers were selected and published in *Soils and Foundations* Journal (Vol. 51, No. 4, 2011). The main themes of the symposium were as:

1. Experimental investigations from very small strains to beyond failure
2. Behavior, characterization, and modeling of various geomaterials
3. Practical prediction and interpretation of ground response: Field observations and case histories

The symposium started at the early morning of the 1st day of September, with an initiative and passionate opening ceremony. Prof. Hong-Taek Kim (Symposium chairman) and Prof. Yeon-Soo Jang (President of KGS) gave a warm welcoming speech as they greeted over 300 participants. Prof. Hervé Di Benedetto (ISSMGE TC101 Chair; France) and Prof. Askar Zhussupbekov (ISSMGE Vice president for Asia; Kazakhstan) delivered congratulatory addresses on behalf of all members of the ISSMGE.

CONFERENCE NEWS (Continued)

IS-Seoul 2011; Fifth International Symposium on Deformation Characteristics of Geomaterials

Opening ceremony. Welcome address by Prof. Hong-Taek Kim (Symposium Chairman).

Gift presentation. Prof. Y.S Jang (President of KGS) and Prof. Askar Zhussupbekov (ISSMGE VP Asia).

Group photograph (after the opening ceremony).

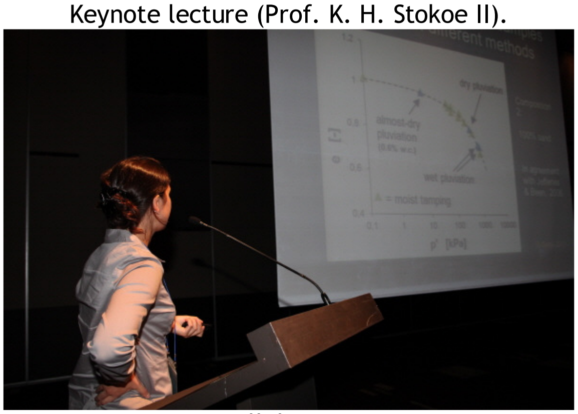
During the intensive three day schedule, 7 invited lectures (including the 1st Bishop lecture sponsored by ISSMGE), 14 technical sessions, and 2 poster sessions were held to provide 167 presentations (97 in oral and 70 posters) to share academic knowledge and discover future opportunities on geotechnical engineering aspects. Each participant actively participated in their presentations, question and answer sessions, and discussions.

CONFERENCE NEWS (Continued)

IS-Seoul 2011; Fifth International Symposium on Deformation Characteristics of Geomaterials

Keynote lecture (Prof. K. H. Stokoe II).

Audiences.



Parallel session.

Poster session.

Among others, the 1st Bishop lecture provided by Prof. Fumio Tatsuoka (Tokyo University of Science, Japan) was the climax of this symposium. The Bishop lecture was presented with the sponsorship of the ISSMGE TC101 to honor the contributions and achievements of Professor Alan W. Bishop in the field of geotechnical engineering. Professor F. Tatsuoka presented significant roles of laboratory stress-strain tests of geomaterials for academic developments in geotechnical engineering research. More than 250 audiences attended this precious and honorable moment.

International Society for Soil Mechanics and Geotechnical Engineering
Société Internationale de Mécanique des Sols et de la Géotechnique

The International Society for Soil Mechanics and Geotechnical Engineering and the ISSMGE Technical Committee 101 on Laboratory Testing

Recognize

Professor Fumio Tatsuoka

as the

FIRST BISHOP LECTURER

and thank him for delivering his lecture entitled

"Laboratory stress-strain tests for developments in geotechnical engineering research and practice"

on Sept. 2nd, 2011 in Seoul, Korea, at the occasion of the Fifth International Symposium on Deformation Characteristics of Geomaterials

THE LECTURE WAS ESTABLISHED IN 2009 IN RECOGNITION OF PROFESSOR ALAN BISHOP

Jean Louis Briaud
President of ISSMGE

Hervé Di Benedetto & Satoru Shibuya
Chairs of the TC 101

Appreciation plaque for the 1st Bishop lecture.

The 1st Bishop lecturer, Prof. F. Tatsuoka.

CONFERENCE NEWS (Continued)

IS-Seoul 2011; Fifth International Symposium on Deformation Characteristics of Geomaterials

One of the biggest satisfactions of this symposium was active and lively discussions among the participants. All sessions and even coffee breaks provided interesting and passionate forum for discussions. Moreover, valuable official dinners were held to share and promote international friendship between scholars from other countries. Special Korean culture shows (e.g. *Samulnori* and Korean fan dance) were performed to add amusements and to deepen understandings on Korean culture.

Welcome reception on September 1st, 2011.

Culture show: Korean percussion, *Samulnori*.

Main Banquet on September 2nd, 2011.

Culture show: With the Korean fan dance team.

After the fruitful three-day symposium, a closing ceremony was held to close the symposium. Prof. Dong-Soo Kim who was the general-secretary of IS-Seoul 2011 summarized the statistics of the symposium and appreciated participation and collaboration of all members. The next international symposium venue was announced to be hold in Buenos Aires, Argentina in 2015 (IS-Argentina 2015). For details, presentation files of invited lectures and symposium photos are opened to the public on the symposium website (www.isseoul2011.org).

Finally, we want to appreciate all symposium participants, members of: the Local Organizing Committee (LOC); ISSMGE TC101; and International Advisory Board, for their contributions and collaborations in order to make the IS-Seoul 2011 a success. Moreover, we return thanks to our major sponsors: National Research Foundation of Korea, The Korean Federation of Science and Technology Societies, Seoul Tourism Organization, and 16 Korean engineering and construction companies for their financial supports.

CONFERENCE NEWS

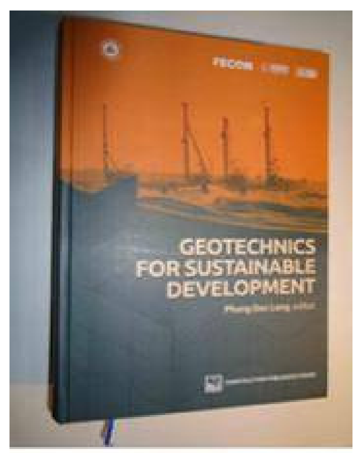
INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Geotec Hanoi 2011, an international conference on Geotechnics for Sustainable Development" was organized by FECON Foundation Engineering & Underground Construction JSC, one of the leading geotechnical contractors in Vietnam, in cooperation with Vietnamese Society for Soil Mechanics and Geotechnical Engineering (VSSMGE), and with Civil Engineering-Mechanics-Material Association, France, in Hanoi for two days of 6th and 7th October 2011. The conference attracted 450 attendees from 24 countries. At the conference 110 papers were published, among which 60 papers/lectures were presented.

The conference had six main themes: 1) Soft soil improvement and reinforcement, 2) Foundation engineering, 3) Tunneling and underground spaces, 4) Environmental geotechnics and sustainable development, 5) Geotechnical modeling, design and monitoring, and 6) Geotechnical case histories. The six sessions were held in parallel in two conference halls.

Among the conference high-lights were the six keynote lectures given by six well-known geotechnical experts in the world:

- Keynote lecture 1: "Soil improvement by preloading and vertical drainage" given by Professor Sven Hansbo (Sweden);
- Keynote lecture 2: "The design of high-rise building foundations" given by Professor Harry Poulos (Australia);
- Keynote lecture 3: "Tunneling in soft ground and urban environment" given by Professor Alain Guilloux (France);
- Keynote lecture 4: "Current facts concerning efforts to improve the global environment and commitments by the construction industry in Japan" given by Dr. Hiroshi Yoshida (Japan);
- Keynote lecture 5: "New horizons in numerical analysis for geotechnical engineering" given by Professor Pieter Vermeer (the Netherlands);
- Keynote lecture 6: "Characteristics of liquefaction-induced damage in the 2011 Great East Japan Earthquake" given by Professor Kenji Ishihara (Japan).



The conference proceeding

The conference proceeding, which has an international registration number ISBN 978 604 82 000 8, was very well edited with international high-quality printing and hard cover. The proceeding consists of 996 pages and includes 110 papers, all written in English, which were divided into six sessions. A CD Rom is also provided, including all the papers in pdf files with color figures/photos. The proceeding can be ordered through the conference website. Contacts address is; Conference website: *http://www.geotechn2011.vn* and mail contact: *secretariat@geotechn2011.vn*; *phung.long@gmail.com.*

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Conference photographs are shown in what follows.

Figure 01. Dr. Phung Duc Long, VSSMGE vice-president and chairman of conference scientific committee, opened the conference.

Figure 02. Mr. Pham Viet Khoa, FECON President and chairman of the conference, made a welcome speech.

Figure 03. Part view of the audience.

Figure 04. Part view of the audience.

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Figure 05. Keynote lecture by Prof. Sven Hansbo (Sweden).

Figure 06. Keynote lecture by Prof. Kenji Ishihara (Japan).

Figure 07. Keynote lecture by Prof. Harry Poulos (Australia).

Figure 08. Keynote lecture by Prof. Pieter Vermeer (Netherlands).

Figure 09. Keynote lecture by Dr. Hiroshi Yoshida (Japan).

Figure 10. Keynote lecture by Prof. Alain Guilloux (France).

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Figure 11. Prof. Nguyen Ba Ke, Chairman of FECON's scientific committee, closed the conference.

Figure 12. Keynote lecture speakers and conference organizers, from right to left: Mr. Pham Viet Khoa, conference chairman, Dr. Phung Duc Long, chairman of conference scientific committee, Prof. Alain Guilloux, Prof. Harry Poulos, Prof. Sven Hansbo, Prof. Kenji Ishihara, Prof. Pieter Vermeer, and Prof. Nguyen Truong Tien, conference co-chairman.

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Figure 13. Handling attendance certificates to selected attendees.

Figure 14. A memorable picture: conference organizers and special guests.

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Figure 15. Gala dinner - A toast for the conference success from the organizers, from right to left: Mr. Bui Nguyen Hoang, conference co-chairman, Prof. Nguyen Truong Tien, conference co-chairman, Dr. Phung Duc Long, chairman of conference scientific committee, and Mr. Pham Viet Khoa, conference chairman.

Figure 16. Gala dinner - Special guests.

CONFERENCE NEWS (continued)

INTERNATIONAL CONFERENCE GEOTEC HANOI 2011 "Geotechnics for Sustainable Development", in Hanoi, 6th-7th October 2011

Fig. 17. Post-conference tour to Ha Long Bay, Prof. Sven Hansbo enjoyed the tour.

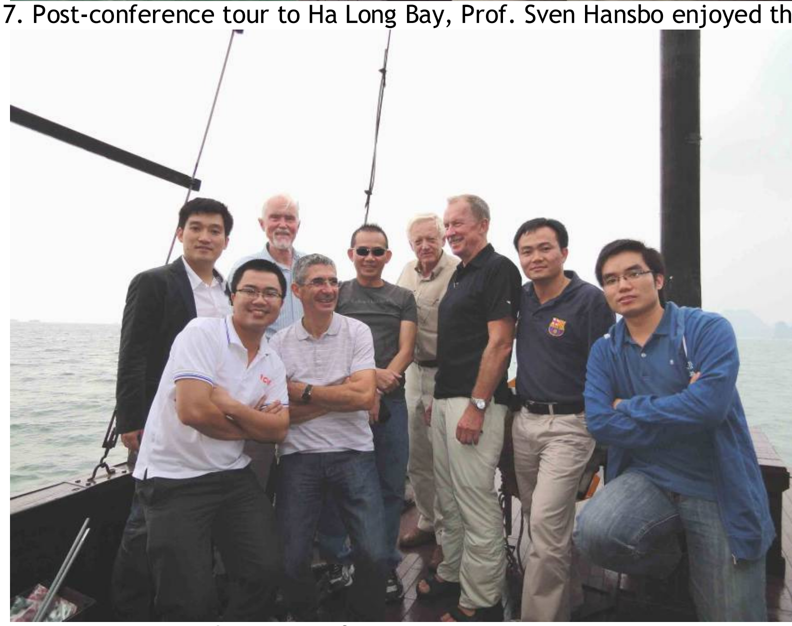


Fig. 18. Post-conference tour to Ha Long Bay.

TECHNICAL NEWS

VISIT OF ATC3 COMMITTEE ON SLOPE INSTABILITY SITES IN BHUTAN

Ikuo Towhata, University of Tokyo, towhata@geot.t.u-tokyo.ac.jp
Mitsu Okamura, Ehime University, okamura@cee.ehime-u.ac.jp
Hirofumi Toyota, Nagaoka University of Technology, toyota@vos.nagaokaut.ac.jp

INTRODUCTION

Asian member societies of ISSMGE have been operating several technical committees of their own and among those committees is ATC3 that concerns geotechnical natural hazards. In the current 4 years of term, this committee is chaired by Ikuo Towhata and is working on slope problems. As a part of the ATC3 activities, three committee members made a visit to Bhutan from October 18th, 2011, to 25th, and carried out some studies in collaboration with the Department of Geology and Mines of Bhutan Government and DHI-Infra Ltd.

Figure 1 illustrates the general idea of the Kingdom of Bhutan which ranges from the lowland at its Indian border to the top of Himalaya. The size of Bhutan is 38,400 km^2 in area and its population is 700 thousands. Because of the tectonic action between the Indian Ocean Plate and the Eurasian Plate, the geology in Bhutan is highly distorted and fractured, which makes mountain slopes highly vulnerable to instability problems. The precipitation rate is 3,000 to 5,000 mm in the southern lowland, 1,200 to 2,000 mm in the lower Himalayan slopes, 500 to 1,000 mm in the central mountain regions, and less than 500 mm in Himalaya. Most precipitation takes place during the monsoon season of June to September.

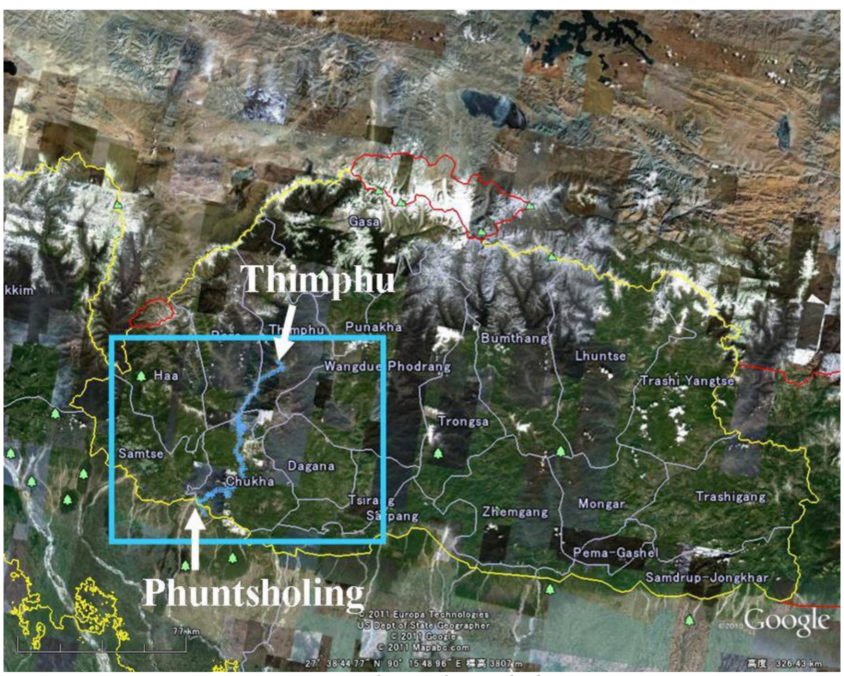


Fig. 1 Map of Kingdom of Bhutan.

TECHNICAL NEWS (Continued)

VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

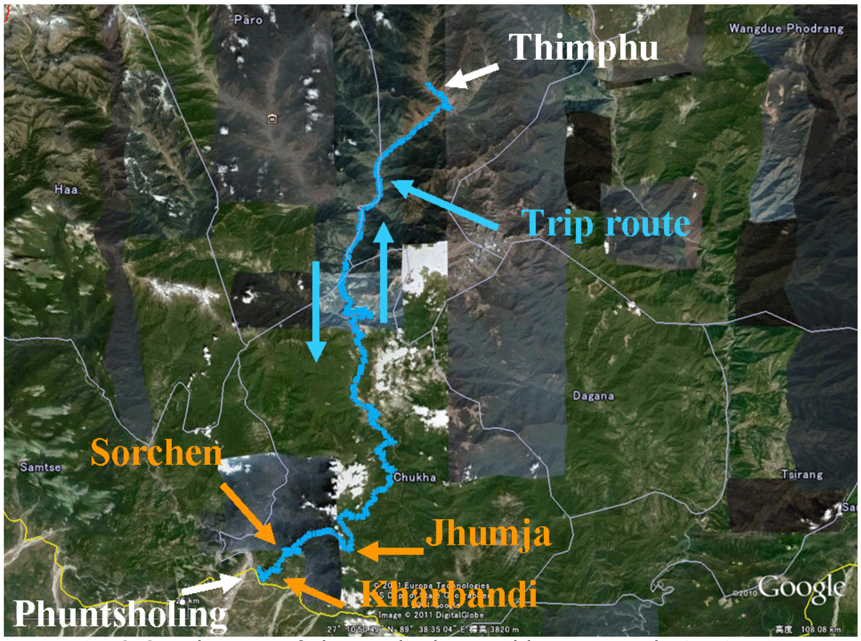


Fig. 2 Google map of the studied area (blue rectangle in Fig. 1).

Among many geohazards in this nation, 25 glacier lakes at high altitude are prone to breaching and flooding. The breaching event in 1994 caused debris flow in the downstream area, and the former capital of Punakha was affected. Although attempts are being made to construct drainage channels, working at high altitude is very difficult and risky. Because of the mission of ATC3, the present study was conducted on slope instability problems in the southern half of the country (Fig. 2).

The capital city of Thimphu has a population of 100 thousands approximately. Thimphu is located in the middle of mountainous region and there is only one major road that connects this city and India from which food, fuel, and other living substances are supplied. Thus, any slope failure along this important road would be fatal to the activities in the capital. Steep mountains and deep gorges along this important road have made road construction extremely difficult.

SLOPE INSTABILITIES

Technical visits were made of many sites of slope instability that is affecting the traffics of the nation's No. 1 highway between Thimphu (capital) and Phuntsholing (second biggest city at the Indian border); see Fig. 3. It appears that such metamorphic rocks as gneiss, schist, and phyllite with significant fissures and weathering are exposed to the air on the mountain side of the road and are causing instability problems at many places. Because the studied highway is the unique connection from outside to the capital, the slope instability is a substantial problem to the nation's economy.

Fig. 3 The highway connecting Thimphu and Phuntsholing.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

Jhumja Site

The road was opened in 1961. Fig. 4 shows that the road has subsided by 6 m over the distance of 200 m or so. The Road Department states that the subsidence occurred between 1998 and 2009, implying that the annual subsidence was about 60 cm. The typical type of rock here is gneiss and other metamorphic rocks.

It appears that this slope movement is caused by a deep-seated sliding of the rock mass, and the consequent distortion of the surface rock mass results in fracturing and stone falls (left side in Fig. 4). Another possible cause of stone fall (Fig. 5) is the mechanical weathering of the surface rock that was probably fractured by blasting construction of the road. It deserves note that major stone fall stopped in 2009 for unknown reasons.

Fig. 4 Jhumja slide.

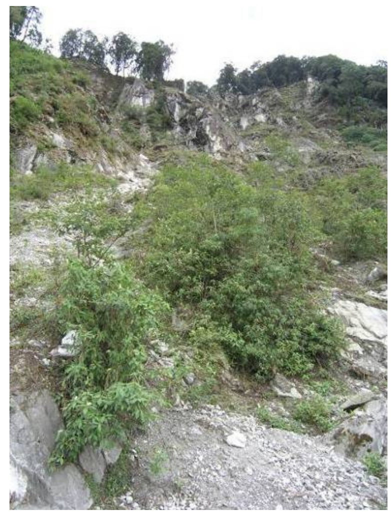


Fig. 5 Rock falls in Jhumja slide.

Sorchen Site

Road is affected by stone falling and slope instability at Sorchen (Fig. 6). The fallen stones deposit in the lower part of the slope (Fig. 7) and are prone to further failure, closing the road, in rainy seasons. The base of the slope consists of fractured Quartzite which is subject to hydration, weathering, and deterioration.

The surface weathering and instability may be mitigated by removal of unstable materials at the slope surface and then covering the surface by shotcrete, thus isolating the rock from external actions. However, this mitigation measure is temporary and the problem may start again in near future. The fundamental problem at the Sorchen site is the geology. Fig. 8 indicates that three slope failures are aligned in a narrow range, suggesting the effects of the regional distribution of vulnerable rock (Quartzite and Phyllite). Because of this reason, the mitigation at the slope surface is nothing more than a temporary action and more fundamental mitigation such as changing the road route is desired.


Fig. 6 Failure of cut slope at Sorchen.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

Fig. 7 Deposit of stones at bottom of Sorchen site.

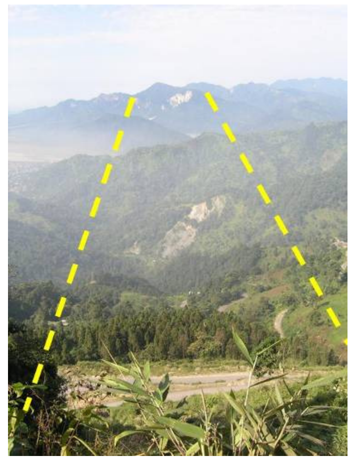


Fig. 8 Series of slope instability to the south of Sorchen site.

Kharbandi Site

The Karbandi site is located immediately behind the town of Phuntsholing and the road is located at the top of a saddle topography. Fig. 9 shows that the head scarp of the eroded cliff is approaching the main road. The rate of erosion is approximately of the order 1 m / year at the slope shoulder, according to the past observation. Fig. 10 indicates the erosion and gully formation. Phyllite forms this valley. At some time in the past, a minor action at the bottom of the valley triggered a small erosion and instability, and then the problem has been developing into a bigger scale. At this moment there is no efficient measure to stop this erosion at a reasonable cost, and the nation’s most important road is going to be in a critical situation.

The local geology is composed of phyllite which is vulnerable to fracturing caused by hydration and reduced overburden pressure (Fig. 11). There are some more sites of similar problem (Fig. 12).

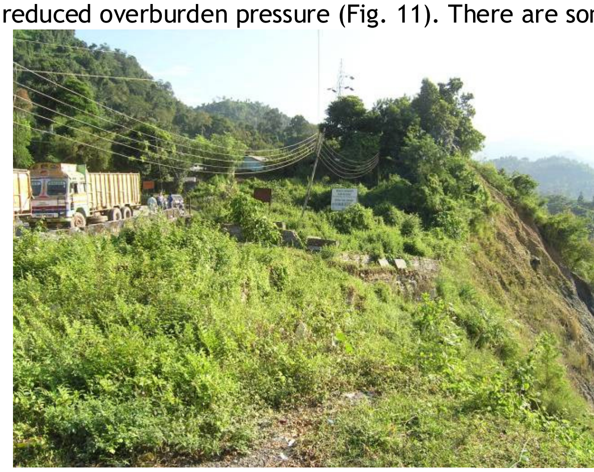


Fig. 9 Karbandi slope subjected to material det erioration and erosion.

Fig. 10 Ongoing erosion at Karbandi.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

Fig. 11 Breakage of Phyllite rock by we athering and water action.

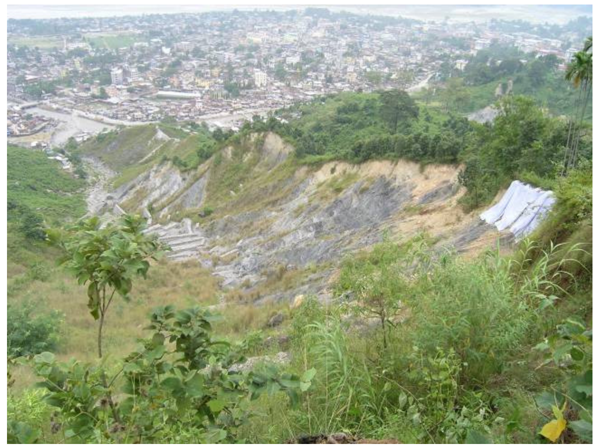


Fig. 12 Instability of Phyllite slope near

Figure 13 illustrates the site of stone falls during an earthquake in neighboring Sikkim on September 18th, 2011, with M=6.9. This cliff was formed in a terrace deposit and is composed of rubbles and stones. After the seismic disturbance, more stones are falling during rains and the road traffic at the bottom is not safe. Currently there is no rule in Bhutan about responsibility for the stabilization of such a slope. It is interesting that some parts of the top of this cliff are stabilized by simple masonry walls, thereby causing no slope instability.

Fig. 13 Seismic falling of debris from cliff of terrace deposit in Phuntsholing.

SEMINAR AT DGM

Prior to the field trip, a half-day seminar was held at the Department of Geology and Mines (DGM), Government of Bhutan. Because of the aim of the visit, the topics of three of us focused on instability problems in natural slopes. A group photo of participants after the seminar is shown in Fig. 14.

Fig. 14 Group photograph of authors and seminar participants at DGM.

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VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

Talk by Ikuo Towhata

This talk addressed the field monitoring and early warning by which people are able to evacuate in advance and fatal disaster is avoided. A secondary disaster during restoration of damage is avoided as well. Because the location of future failure in a natural slope is difficult to foresee due to complicated variation of local slope angles, types of rocks, extents of weathering, and local hydrology, it is more desirable to install many monitoring sensors at any suspected parts of a slope than to pursue accurate observation. Thus, the costs for manufacturing of a sensor and its installation as well as operation have to be low.

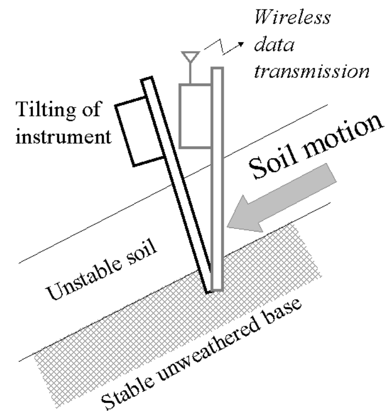


Fig. 15 Working mechanism of tilting sensor.

The authors' attention has been focused on rainfall-induced quick failure of surface unstable materials in slopes. Surface failure is small in size but many in number and affects people's life and property all over the world. In contrast, a deep-seated large slope failure is not necessarily the target of study.

The proposed inexpensive sensor is intended to monitor movement of the surface unstable layer during heavy rainfall. For its working mechanism, see Fig. 15. Former studies (Farooq et al., 2004, and Orense et al., 2004) showed that soil exhibits minor deformation prior to the rain-induced failure. Thus, the developed sensor monitors deformation of slopes during heavy rainfall. The sensor is placed at the top of a rod which is penetrated in advance into ground until reaching a stable base layer. When the surface weathered soil starts to move during rain, the rod tilts and the tilting angle is monitored by the sensor. The recorded data is sent through wireless to an office and, if the extent of the data exceeds a threshold, caution or warning is sent to the local community.

The Three Gorge Dam in China produced a huge reservoir and the rising of the water level caused slope instabilities along the lake. The proposed tilting sensors were installed at one of such sites (Fig. 16). The slope had been distorted already upon the installation.

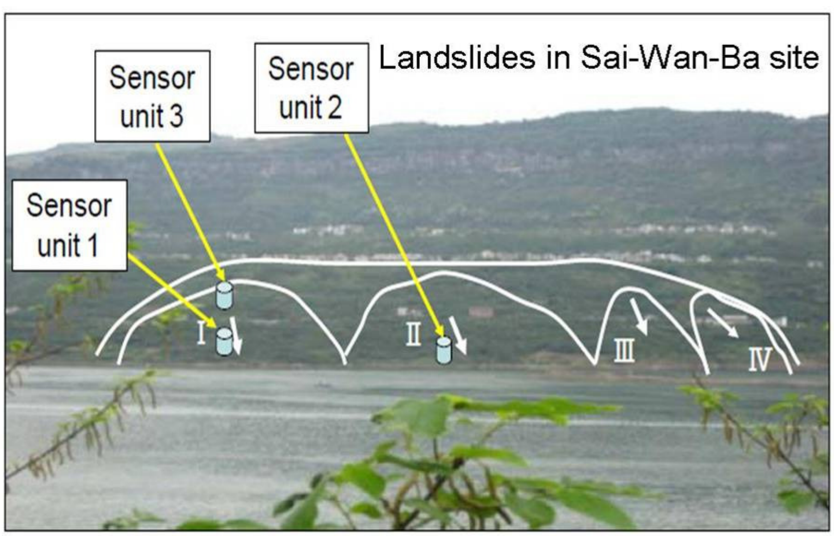


Fig. 16 Validation site at Three Gorge Dam Reservoir in China.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

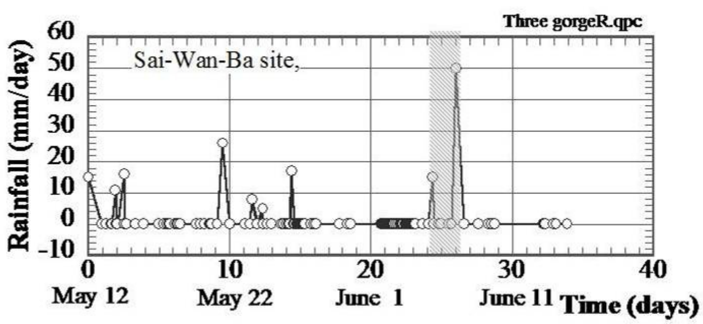
One of the monitored slopes failed on June 7th and 8th, 2009, during rain (Fig. 17). Fig. 18 shows the monitored records of tilting angles and precipitation. It is seen that the final failure (large angle) was preceded by minor rate of ground distortion. By comparing the recorded rate of tilting with the current proposal of threshold rates of tilting;

Caution if rate of tilting angle > 0.005 degree / hour, and
Alert / Evacuation if the rate > 0.1 degree / hour,

it is found that these warning thresholds are consistent with the records. However, it is thought that the threshold for the first caution is still subject to discussion.

Fig. 17 Failed slope at Three Gorge Dam Reservoir

(a) Rainfall records



(b) History of tilting angle

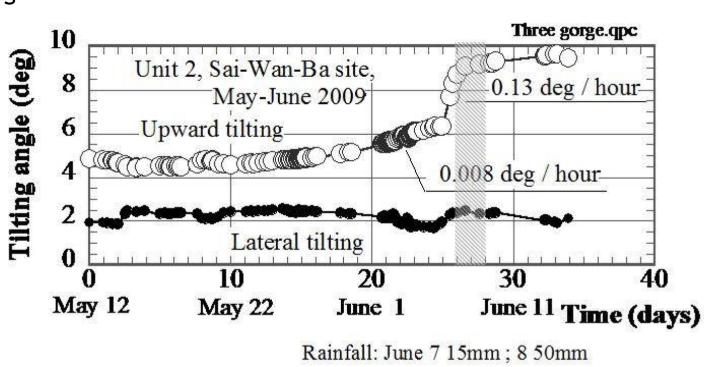


Fig. 18 Monitored records at the Three Gorge Dam Reservoir.

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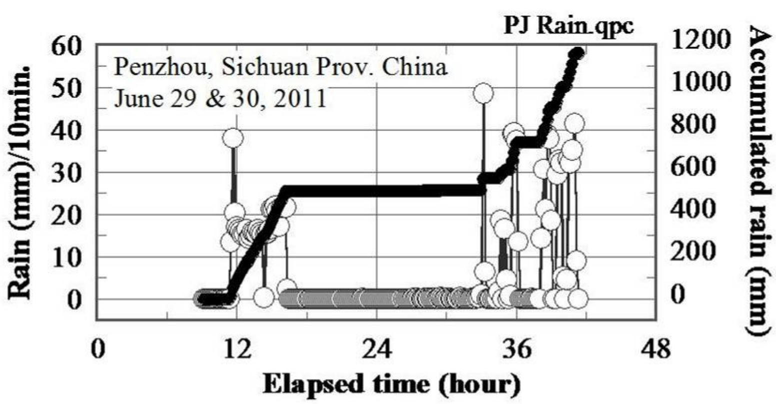
An artificial rainfall test was conducted in Sichuan Province of China in June 2011 in collaboration with the Chengdu Institute of Mountain Hazards and Environment. Fig. 19 illustrates the ongoing artificial rainfall, and finally an excavated trench face fell down (Fig. 20). The obtained record in Fig. 21 demonstrates that the rate of tilting angle is consistent with the proposed threshold as mentioned above.

Fig. 19 Artificial rainfall on slope in Sichuan Province, China.

Fig. 20 Rainfall-induced failure of a cut slope.

(a) Rainfall



(b) tilting angle

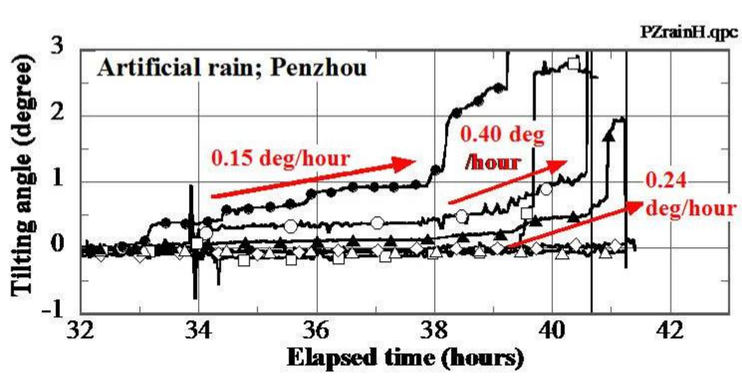


Fig. 21 Monitored records during artificial rainfall.

TECHNICAL NEWS (Continued)
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To date, many early warning projects monitor only rainfall. The rainfall threshold to issue warning is relatively easy, but it relies on empirical correlations and may not be good enough for a particular slope subject to locally different slope angles and geology. In this regard, monitoring any movement directly is more promising and should be utilized in combination with the rainfall threshold.

Talk by Mitsu Okamura

Fig. 22 Retaining wall sliding down with road embankment.

An attempt was made to interpret the seismic stability of road embankments resting on sloping ground with retaining walls (Fig. 23). For pseudostatic calculation of stability, appropriate values of seismic coefficient were assessed by using an empirical correlation with the JMA scale of seismic intensity. The bearing capacity of a retaining wall was determined simply on the basis of cone penetration tests. As a consequence, Fig. 24 illustrates that the extent of damage is well correlated with the factor of safety greater than or less than unity. Thus, the use of seismic factor of safety with a CPT correlation of soil strength and pseudostatic seismic action is meaningful in evaluation of damage extent in the concerned type of structures.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

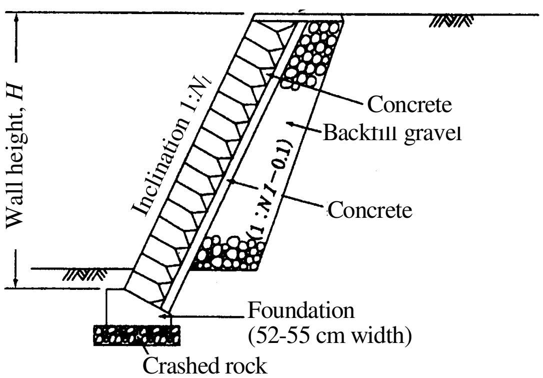

Fig. 23 Standard cross section of masonry retaining wall.

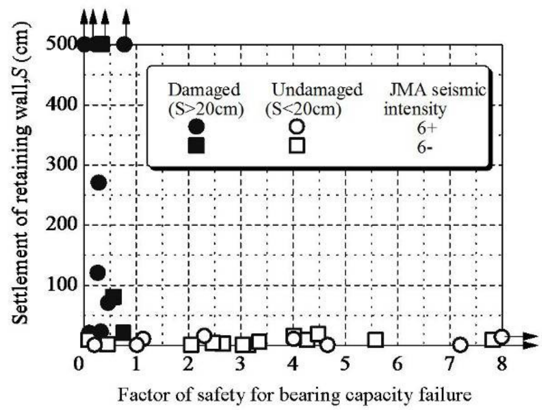

Fig. 24 Relationship between settlement of retaining wall and factor of safety.

Talk by Hirofumi Toyota

This talk addressed the observation of slope failures during natural disasters that occurred in the Chuetsu area of Niigata, Japan.

Many natural disasters, such as earthquakes, heavy rainfalls and snowfalls, occurred in the recent times in the Chuetsu region of Niigata, Japan, between 2004 and 2011. Slope failures during the time are summarized here and, in particular, the progress of the slope damage is examined, from the perspective of compound disasters. Further, the importance of local geology force and ground water condition is stressed as the reasons for the occurrence of numerous landslides during the 2004 Chuetsu Earthquake

Figure 25 illustrates a geological map of the severely damaged area during the Chuetsu Earthquake. More than 3,000 slope failures occurred during the earthquake. Although there had been many landslide-prone areas in the massive mudstone area, most slope failures during the earthquake occurred in the sandstone-dominated area despite that there had been only a small number of the landslide-designated slope therein.

Figure 26 shows the general behavior of ground water level in the concerned area. Ground water level was high at the time of the Chuetsu earthquake because of the typhoon rain a few days before. In general, groundwater level rises during the snow-melting season and suddenly drops after snow melting in May. Hence, landslides frequently occur in April and May. However, large mass movement did not occur significantly during the snow-melting season after the quake, probably because the seismically-affected slopes became stable after the quake and the seismically-induced deformation.

TECHNICAL NEWS (Continued)

VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

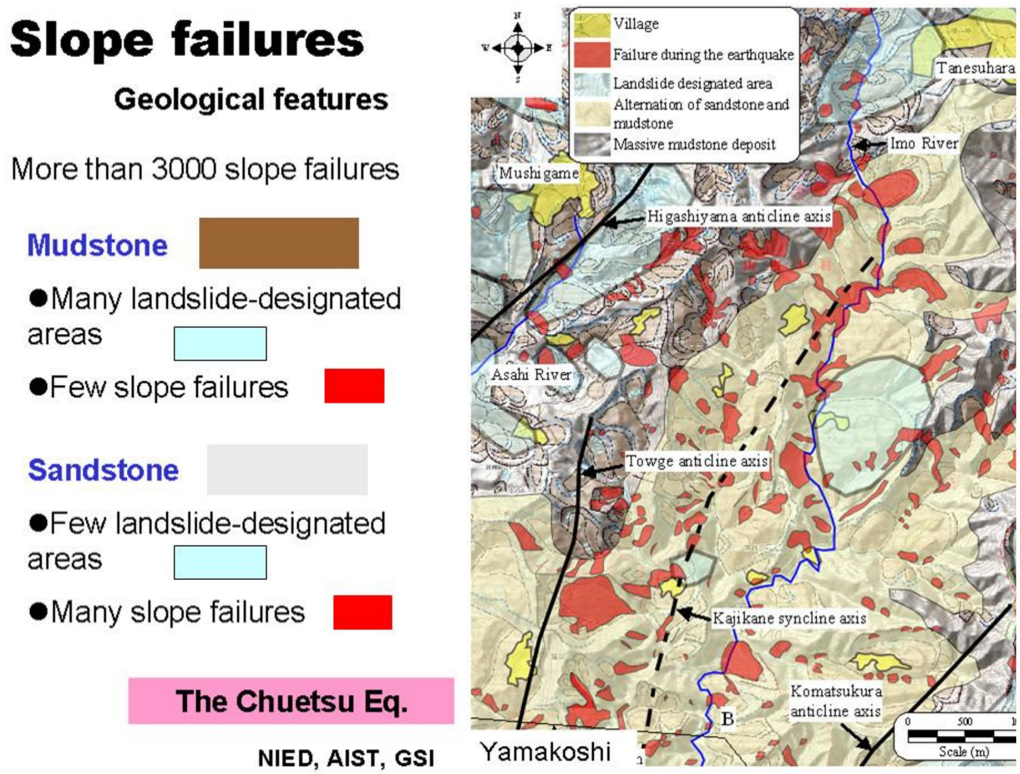

Fig. 25 Slope failures during the Chuetsu Earthquake on simplified geological map.

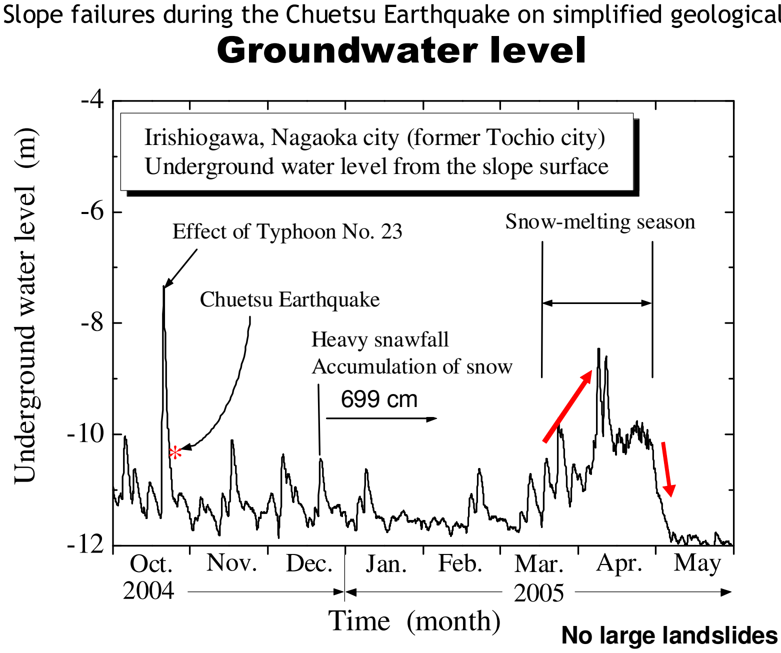

Fig. 26 Ground water level measured in an old landslide area.

TECHNICAL NEWS (Continued)
VISIT OF ATC3 COMMITTEE TO SLOPE INSTABILITY SITES IN BHUTAN

ACKNOWLEDGEMENT

The present ATC3 visit to Bhutan was made possible by contribution of Dr. Karma of DHI-Infra Ltd. and Mr. Samten Wangdi of DGM The development of tilting angle sensor has been made by Dr. T. Uchimura of University of Tokyo and Dr. Wang Lin of Chuo Kaihatsu Corporation. The field validation tests in China were conducted in collaboration with Prof. J.P. Qiao and his colleagues at the Chengdu Institute of Mountain Hazards and Environment, China. These efforts and contributions are deeply appreciated by the authors. Last but not least, after all technical works the authors visited a classic *dzong* (compound of religious and administrative institutes) in a local town of Wangdi Phodrang (Fig. 27). The well preserved building in this dzong was one of the highlights of the visit to Bhutan. It is desired that the good tradition of this country will last forever.

Fig. 27 Wangdi Phodrang Dzong.

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Case History
Submarine Landslides on the South-Eastern Australian Margin

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1. INTRODUCTION

The consequences of submarine landslides include damage to seabed infrastructure (communications cables and buried pipelines), subsidence of coastal areas and the generation of tsunamis (Masson *et al.*, 2009). Our failure to understand the causes of these phenomena means that submarine landslides present a significant risk to coastal and offshore development, and have on occasion resulted in the halting of offshore developments. It has been established that large submarine landslides can produce tsunamis, such as the earthquake triggered submarine slides in 1929 (Grand Banks, USA; Fine *et al.*, 2005) and 1988 (Aitape, Papua New Guinea; Tappin *et al.*, 2001) which both resulted in significant casualties. The large loss of life and damage to infrastructure from the Indian Ocean tsunami of 2004 (Lay *et al.*, 2004) and the recent Japanese event have increased interest in the tsunamigenic potential of large submarine slides. Australia is vulnerable to tsunamis with 85% of the population living within 50 km of the coast and much of the critical infrastructure located close to the coast. It has been suggested (Dominy-Howes, 2008) that the maximum credible tsunami could cover mainly in 6 m of water, and while the possibility of such an event has major implications for risk assessment and siting of critical infrastructure, the likelihood cannot be sensibly determined.

The geological record contains many examples of submarine landslides, which can vary in scale from minor shallow slides to very large slides, such as the Storegga slides off the Norwegian coast which have a total volume of over 3000 km^3 (Haflidason *et al.*, 2005). Statistics on known landslides on the eastern continental slope of North America, which has geological similarities to Australia's eastern margin, have been published by Masson *et al.* (2006). These show that between 30^{o}N and 45^{o}N there are 152 large landslides affecting an area of nearly 40000 km^2. Most failures occur on slopes of between 1^o and 7^o, with the greatest number of failures occurring on slopes of 3.5^o. The area affected by failures decreases as the slope increases, and the depth of water at the slide head ranges from 250 m to 2500 m, with the greatest number of failures occurring at water depths of around 1000 m. Despite extensive literature on the nature and causes of submarine landslides, their dynamics and triggering processes are not well understood (Locat and Lee, 2002; Bardet *et al.*, 2003). One of the principal reasons for this is the limited data on the physical and mechanical properties of the sediments, particularly from the slide plane, as these materials have not traditionally been collected.

In Australia, studies of the southeastern (SE) Australian continental slope (Jenkins and Keene, 1992; Glenn *et al.*, 2008; Boyd *et a*l., 2010) have been very limited until recently. Evidence of submarine landslides on the SE Australian margin was first reported by Jenkins and Keene (1992), but it was not until high resolution, multi-beam bathymetric data became available (Glenn *et al.*, 2008; Boyd *et al.*, 2010) that the true distribution of these slides could be established. The recent collection of high-resolution multibeam echo-sounding and sub-bottom profiling data has provided a detailed view of mass-transport features over a 900 km length of the margin. A wide range of slide features has been detected as well as a series of canyons which cut through the slope sediments. The submarine slides range in scale from hundreds of small slides with volumes of <0.5 km^3 up to the largest documented slide which has a volume of 20 km^3 (Boyd *et al.*, 2010).

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

2. SUBMARINE LANDSLIDES

2.1 SOUTHEASTERN AUSTRALIAN MARGIN

The SE Australian continental margin stretches 1500 km north from Bass Strait to the Great Barrier Reef (Boyd et al., 2004). The margin which is by world standards narrow, deep and sediment deficient, was formed by rifting in the Cretaceous period between 90M and 65M years ago (Gaina *et al.*, 1998). Since then, margin subsidence has been relatively minor. The continental shelf ranges between 14 and 78 km wide and is relatively flat with a thin sediment cover. The sediment reaches a peak thickness of about 500 m at the edge of the shelf, which occurs at depths ranging between 55 m and 180 m. The continental slope is the region from the shelf edge to the Tasman Abyssal Plain where the water depth is around 4500 m. The continental slope ranges from 28-90 km wide and has average slopes in the range from 2.8° to 8.5°. The sediment cover generally reduces from the shelf edge to the Abyssal plain, and is absent from the lower slope off southern NSW (Boyd *et al.*, 2004).

Figure 1 shows the regions where detailed bathymetric studies have been conducted in the last 5 years and from which the sediments have been recovered that are discussed later. Two ship surveys have been conducted, one of the continental slope off Brisbane (SS2008-12), and the other off Sydney (SS2006/10). The surveys consisted of both sub-bottom profiles and echo-sounder records to provide a detailed picture of the seafloor and reveal the underlying geology. In addition 26 gravity cores were obtained from these regions from areas within and adjacent to several slide features, and further sediment was dredged from deeper water. An overview showing the bathymetry for both of the studied areas is given in Figure 2. At this scale it is possible to see that a number of canyons cut into the slope sediments and most of these are off the major rivers. Further details of some of the slides are shown in Figures 3 and 4.

Close inspection of the bathymetric data reveals several distinct large sediment slides varying in volume from <0.5 km^3 to 20 km^3 on the upper slope (water depths < 1200 m) of the SE Australian margin (Boyd *et al.*, 2010). The large slides typically comprise a distinct U-shaped trough in cross-section (3-6 km wide and 20-250 m deep) backed by an amphitheatre-shaped crestal zone. This slide morphology is similar to the classical circular failure profile described by Varnes (1978), but they are elongated in longitudinal profile. The sides and head walls of the scarps are relatively steep with slopes of up to 17°. The largest slides are the Bulli (Figure 3c) and Shovel Slides, near Sydney, on slopes of around 4.5° that are up to 13 km long and 5 km wide with volumes of 20 km^3 (Glenn *et al.*, 2008) and the Byron slide (Figure 3b), off Byron Bay, with a volume of 3 km^3 and slope of 6.5° (Clarke *et al.*, 2011). Sub-bottom profiling data from multiple sections across the continental slope and in particular across the slides show the sediment is built up of well stratified beds (Glenn *et al.*, 2008), which have been suggested to be evidence of past slide events. In most locations, sediments derived from the slides cannot be detected on the slope and it appears that the slide material has been transported to the abyssal plain. However, in a small number of locations (Figure 3a) where the slopes are less steep (< 2°), the slide debris flow deposits have remained on the slope and contain blocks up to 350 m wide and 50 m high.

Figure 2 shows a large number of canyons that cut into the continental slope sediments. These have been categorised into large box canyons, and smaller narrow linear canyons. The 46 large box canyons are on average 14 km wide, 20 km long and over 600 m deep. They stretch from the middle slope to the abyssal plain, and have slopes up to 17° on the walls. Narrow linear canyons occur in the upper slope sediments, most located in central NSW off major river systems such as the Shoalhaven, Hunter and Tweed or off Fraser Island. Well developed examples are 800-1900 m wide, 120-320 m deep and extend downslope for 14-22 km. Canyon wall slopes are up to 34°, the steepest slopes found on this margin (Boyd *et al.*, 2010).

Case History (Continued)

Submarine Landslides on the South-Eastern Australian Margin

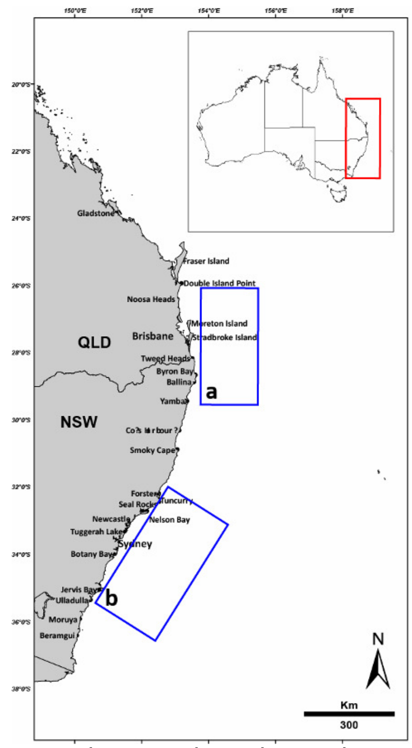

Figure 1: Location map of the two study areas along the southeastern Australian coastline. Blue insets a) and b) mark the location of the bathymetric maps presented in Figure 2.

Further observations from the bathymetry include the widespread slope failures on the mid-slope, shown in Figure 4, which demonstrates an average slope of around 8^o and the widespread relatively shallow failures observed on the Nerang plateau shown in Figure 3a, where the average slope is < 2^o. There are also circular depressions, referred to as pock marks, off Newcastle which are believed to be associated with gas leakage from the underlying Permian coal measures (Glenn *et al.*, 2008).

The figures reveal evidence of widespread erosional features on the SE Australian continental slope. This is different from other margins of similar age, for example the US Atlantic and Gulf Coasts, where sediment deposition is the more dominant process. However, both margins exhibit extensive slides and other erosional features. The 500 m thickness of the sediments on the SE Australian margin has been taken as evidence of a previous period of deposition (Davies 1979, Boyd *et al.*, 2004), but the sediment thickness is substantially less than other margins. This can in part be explained by the dryness of the Australian continent, the relatively subdued highlands and its small rivers. When the resulting low sedimentation is combined with ongoing subsidence of the abyssal plain caused by initial crustal thinning and later thermal cooling, which has caused the gradients on the margin to slowly increase, the result has been a retrogressive gravity failure over all of the lower slope and much of the upper slope. Thus it is considered that the present state of sediment instability, where the overlying sediment wedge is continually undercut by slope failure over geological time, is the cause for modern episodes of failure (Glenn *et al.*, 2008).

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

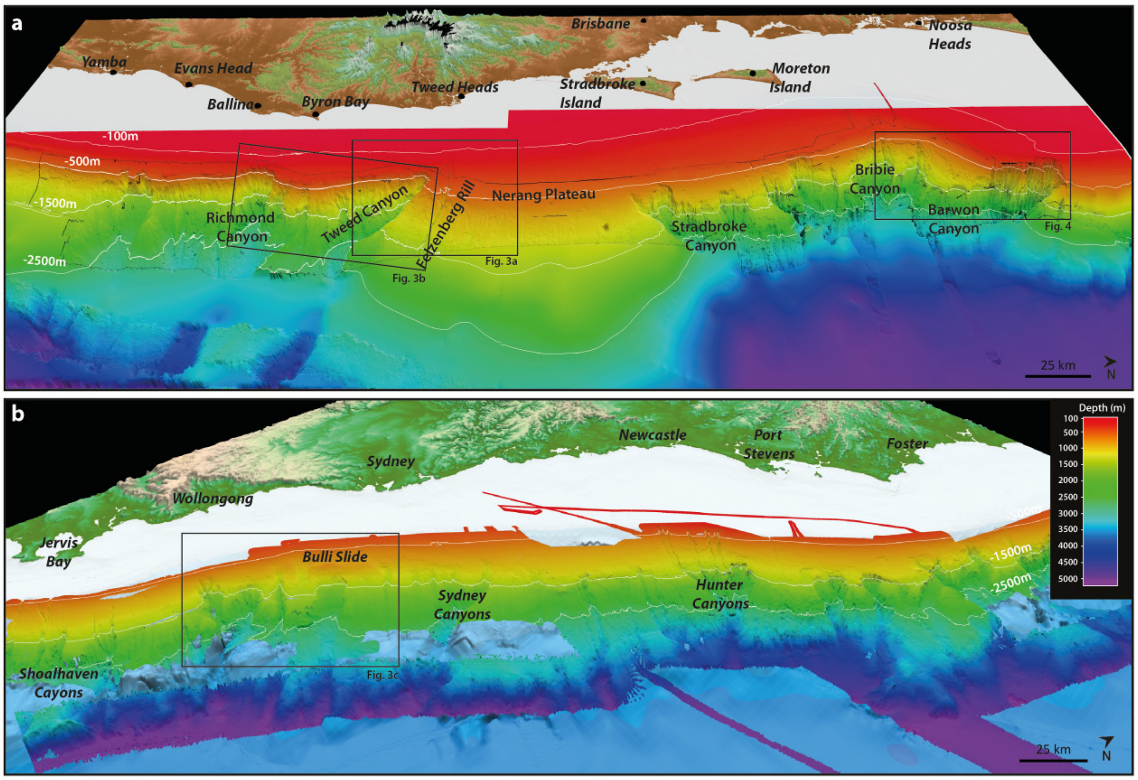

Figure 2: Bathymetric maps of the a) the southern Queensland / northern New South Wales continental slope and b) the mid New South Wales continental slope. Data for these maps were collected on two RV Southern Surveyor voyages: SS2008/12 off the southern Queensland / northern New South Wales coastline (Boyd *et al.*, 2010) and SS2006/10 off the mid New South Wales coastline (Glenn *et al*., 2008). Insets mark the location of the close-up slopes images presented in Figure 3 and Figure 4.

2.2 TRIGGERS

The literature on submarine landslides summarised by Masson *et al*. (2009) and Locat and Lee (2002) lists a variety of causes for their initiation. These include: earthquakes, storm wave loading, erosion and in particular slope over-steepening, rapid sedimentation leading to under-consolidation, the presence of weak layers, gas hydrate dissociation, sea-level changes, glaciations/isostatic uplift, volcanic activity and diapirs. It is also widely accepted that a combination of these factors is required to initiate a landslide, especially where these occur on very shallow slopes. There is data indicating that several large landslides have coincided with earthquakes (e.g. Tappin *et al*., 2001; Bardet *et al*., 2003; Masson *et al*., 2006; Synolakis *et al*., 2002). The role of weak layers, oriented parallel to the sedimentary bedding, has long been used to explain the scale of some large slides, but more recently the importance of weak layers in controlling sliding at all scales has been noted (Masson *et al*., 2009). Despite this Masson *et al*. (2009) also commented that “we know very little about the nature and characteristics of these weak layers, since they have rarely been sampled and very little geotechnical work has been done on sediments recovered from them”. An important consideration is the brittleness of the sediments. Weak layers need to lose strength rapidly and pore pressure needs to rise for effective stresses to reduce. Masson *et al*. (2009) suggest that clay rich sediments with high water content and high plasticity are required for this to occur.

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

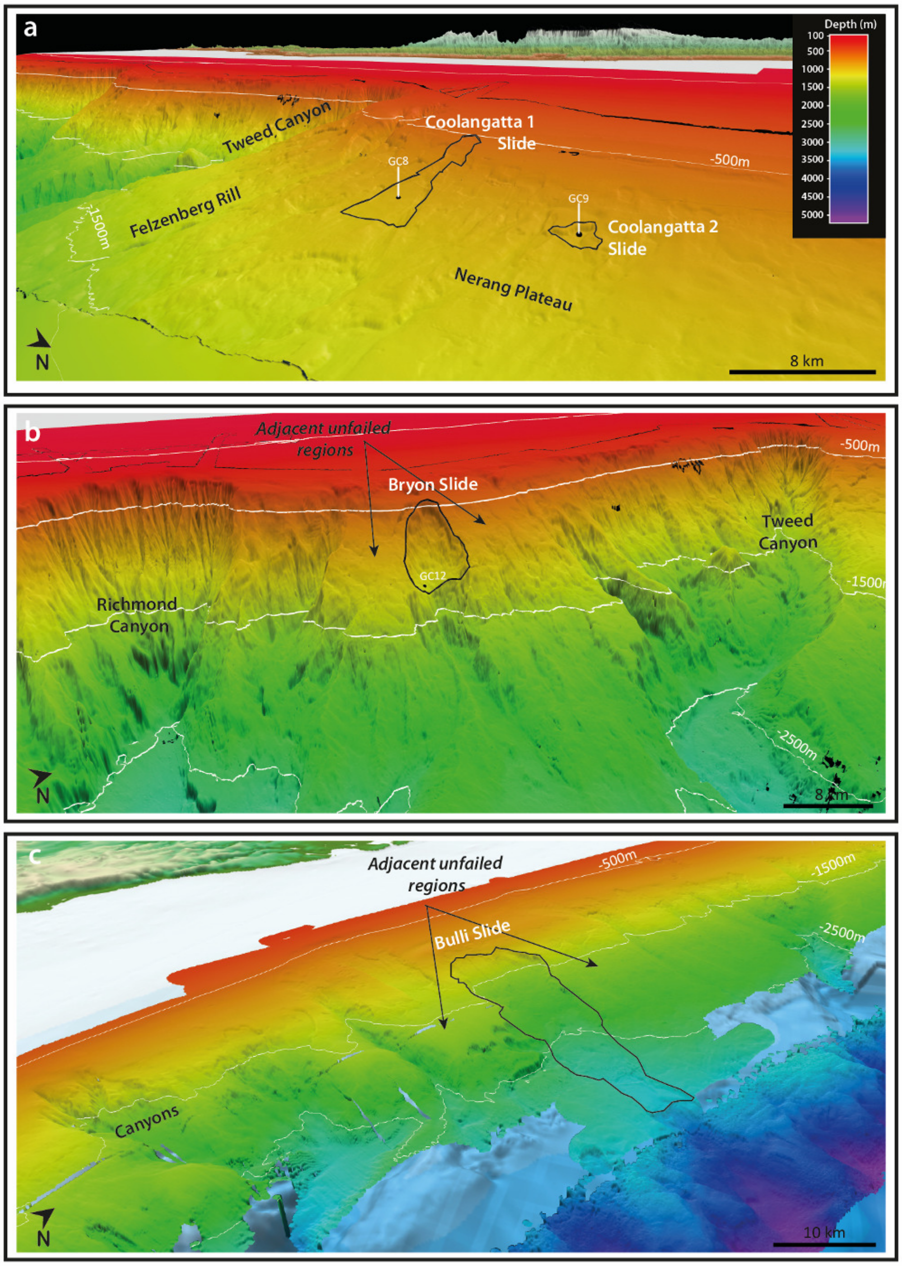

Figure 3: Digital elevation model (DEM) of the slope geometry for four slide sites (outlines denoted by black line): (a) Coolangatta 1 and Coolangatta 2 Slides, (b) Byron Slide, (c) Bulli Slide. Also shown are locations of the three gravity cores (GC8, GC9, GC12) referenced in this study, collected on the RV Southern Surveyor SS2008/12 voyage.

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

Lee (2009) has shown that landslides were more frequent during and just after the last period of glaciation than they are today. One of the suggested reasons for this is that glacial periods coincide with periods of low relative sea level. Figure 5 shows the relative sea level curve for Australia for the last 0.5 million years, which indicates that the sea level has been over 100 m below its present level on several occasions, and the times of these minima are associated with glacial periods. The lowered sea level can increase the likelihood of sliding because it results in the shoreline migrating closer to the shelf edge, leading to increased erosion and higher rates of sedimentation offshore, which now occurs directly on the slope. The lower water pressures (and possibly changed temperatures) can lead to release of gas from gas hydrates increasing pore pressures and reducing strength, and related changes to stress levels in the crust can increase seismic activity. Increased groundwater flows from underlying rocks can occur also contributing to reduced strength. It has also been suggested that changes to deep ocean currents are associated with glaciations and erosion from these currents can contribute to slope steepening (Hubble *et al.*, 2011).

Another mechanism that has been postulated to explain submarine slides is that of creep, slow down slope movements due to gravity stresses that may lead to failure of the sediment mass or to failure on a weak layer at depth. It has been demonstrated that thick deposits on steep slopes can fail by this process (Silva and Booth, 1984). However, as noted by Hampton *et al.* (1996) proof that creep is significant on continental slopes is elusive.

Observations of the widespread occurrence of submarine slides suggests that weak clay layers cannot be a major cause, and tend to favour earthquakes as the triggering mechanism. Nevertheless, it is widely accepted that neither the submarine sliding process nor the slide triggering mechanisms are very well understood (e.g. Locat and Lee, 2002; Mosher *et al.,* 2009), and this is particularly so in the cases of submarine mass failures that do not appear to be linked to seismic activity.

2.3 Sediment Properties

From the recent ship cruises 26 gravity core samples have been obtained, 14 from the region off Sydney and 12 from the region off Brisbane. For most of the gravity cores the soil has been logged and basic properties, particle size distribution, mineralogy and densities have been obtained. The results from the Sydney region have been reported in detail by Glenn *et al.* (2008) and only typical results are reported here. The basic classification tests have been supplemented by a limited number of triaxial, oedometer, shear box and vane shear tests to investigate the mechanical behaviour of the sediments. The mechanical tests have been performed to investigate the landslide triggering processes and in particular to determine the collapse potential of the sediment and the influence of composition and stress level on this behaviour.
A summary of the classification data, which is limited to material from the upper 5.3 m of the sediments as this was the maximum depth of penetration of the gravity corer, is included in Table 1. This shows that the continental slope sediments are predominantly comprised of silt sized material, with about 15% clay, variable amounts of sand sized particles and significant organic content. The sediments contain a significant amount of carbonate grains derived from the remains of living organisms and also significant amounts of terrigeneous material, believed to be transported by the wind from the interior of the continent. Although there is some variability in the composition from core to core there is a broad similarity in the particle distributions all along the margin.

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

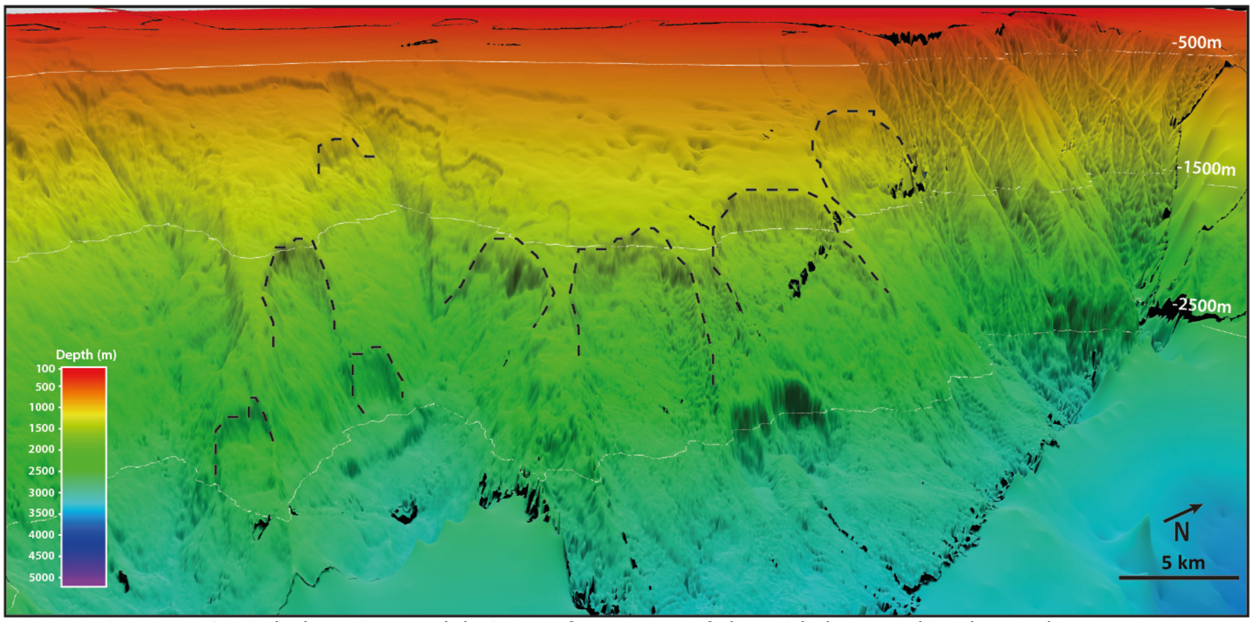

Figure 4: Digital elevation model (DEM) of a section of the mid-slope within the study area. Note the abundance of slump/slide scars presenting arcuate crests (crest outlines denoted by black dashed lines). Modified from Hubble, 2011.

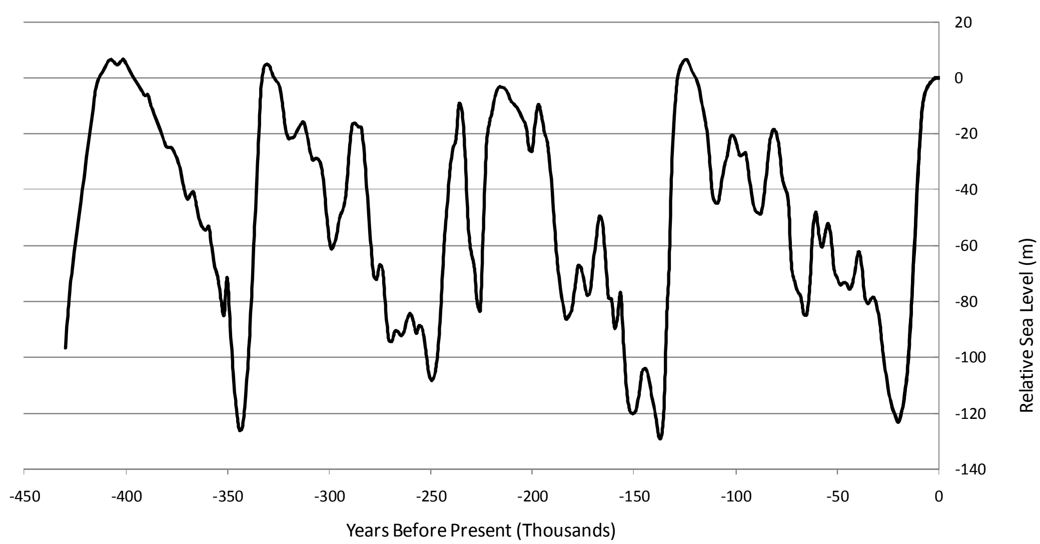

Figure 5: Relative sea level history (after Waelbroeck *et al.*, 2002)

Case History (Continued)

Submarine Landslides on the South-Eastern Australian Margin

Table 1: Summary of available classification data

	Clay (%)	Silt (%)	Sand (%)	Carbonate (%)	Organic (%)	LL	Ip	Moisture content (%)
Brisbane	10-20	50-65	15-40	20	8	46.5	9	50-100
Sydney	8-25	30-80	10-60	35	?	44	18	55-85

Several of the gravity cores were obtained from within detected landslide features in an attempt to penetrate through the base of the slides to assist in constraining the slide depths and dates. In most locations this was unsuccessful as the recent sediment drape overlying the slide surface was thicker than the gravity corer could penetrate. However, in three locations off SE Queensland a distinct boundary in the sediments could be detected at depths between 87 cm and 220 cm. The sedimentological and geotechnical properties of the sediments and their variation with depth from one of these cores (GC12; see Figure 3b for core location) are shown in Figure 6. It can be seen from the figure that there is a distinct change in density and moisture content, as well as appearance of the material at a depth of 87 cm. It is also noticeable that this change in density is not associated with any significant change in the grading, carbonate content or organic content of the material.

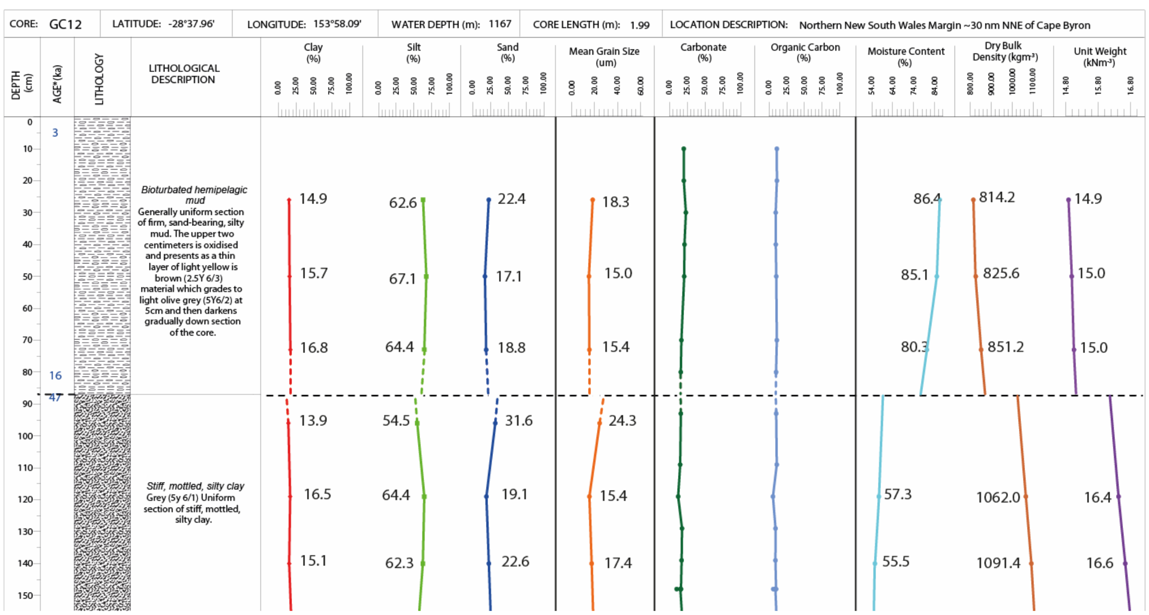

Figure 6: Characterisation of sediment from core GC12, showing physical properties with depth below seabed. Bulk radiocarbon dates are also shown. The presumed slide plane is indicated with a dashed black line at 87 cm depth below seabed (Modified from Clarke *et al*., 2011)

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

Additionally, a bulk radiocarbon (14C) age was determined for sediment sampled directly above the slide plane for this core, returning a date of 15.8 ka for the recent material just above the inferred slide surface (Clarke *et al.*, 2011). This date is consistent with sliding occurring around the time of the most recent sea level low shown in Figure 5. Dating for the deeper sediment could not be determined because its age exceeded that for which 14C dating is reliable. The dating has also enabled the rate of sedimentation to be determined, providing values between 0.3 and 1.2 m/10,000 years. As the rate of deposition is expected to have been higher in the past, these rates of sedimentation suggest that sliding must have been a geologically common event since the formation of the margin 60 million years ago as the current sediment deposit is less than 500 m thick.

Using the values of Cc given below, the change in moisture content at the inferred slide plane can be interpreted as representing a slide depth of anywhere between 10 m and 200 m. The depth reconstructed at the GC12 site by replacing the material apparently missing from the U-shaped trough, i.e. by maintaining the continuity and shape of the adjacent slope and projecting it above the GC12 site, is approximately 250 m. Thus while it is possible to date a possible slide at approximately 16,000 years there is insufficient information to determine whether this is the date of the main slide at this location and further mechanical and dating studies are in progress to further constrain the result.

2.4 GEOMECHANICAL BEHAVIOUR

Limited geomechanical data are available from the vicinity of the slides close to Sydney and from the region off Byron Bay. This has consisted primarily of one-dimensionally consolidated undrained triaxial tests, with some additional shear box tests to evaluate residual strength properties for the Sydney sediments.

Figure 7 shows typical compression plots from 1-D compression tests on undisturbed specimens. Results of three specimens from one of the cores (GC9; see Fig 3a for location), from SE Queensland are shown together with a typical specimen from a core off Sydney. Although there is some variability in the responses the similarity of the response of the specimen from Sydney and SE Queensland is remarkable. Based on these very limited data it appears that the grading, mineralogy and compressibility of specimens on the continental slope are similar along most of the 1500 km of the SE Australian margin. The specimens show high compressibility with Cc values ranging from 0.3 to 0.65. This is considerably higher than would be expected from their remoulded index test results, as the correlation Cc = IpGs/2 would suggest Cc values of 0.13 to 0.26 for plasticity indices of 10% to 20%. It can also be noted that the moisture contents in the upper 5 m are significantly higher than the liquid limit and vane shear tests have indicated that the specimens have significant sensitivities (>2). These data indicate that the slope sediments are structured and, while the cause of the sensitivity has not been established, it could be related to the relatively high organic content of up to 8%, which is known to be a factor in sensitivity in other soils.

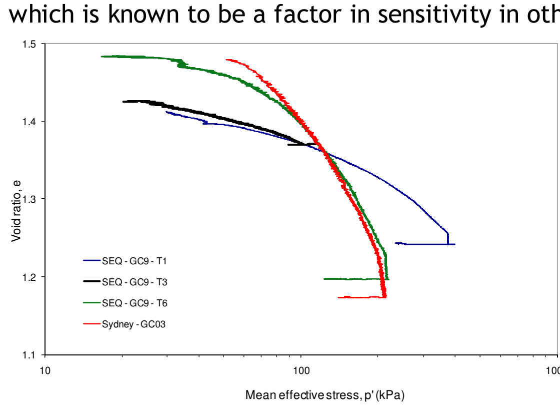

Figure 7: Response of core specimens to 1-D compression

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

The responses of 3 specimens to undrained shearing in triaxial compression are shown in Figure 8, and the associated effective stress-strain curves are shown in Figure 9. To enable comparison of the tests the deviator stress and excess pore pressures have been normalised by the vertical effective stress at the start of shearing. Specimens GC9-T1 and GC9-T3 were adjacent specimens and have similar compressibilities, as seen from Figure 7, but they responded differently to shearing. Specimen GC9-T1 at the higher stress level ($\sigma'v$ = 620 kPa) shows a more brittle response with the peak deviator stress attained at a very small strain, after which the resistance rapidly decreases to its ultimate value. In contrast, specimen GC9-T3 at the lower stress ($\sigma'v$ = 167kPa) does not reach a maximum until relatively large strain. From the pore pressure responses and the effective stress paths it can be seen that this difference is a consequence of a transition from dilative to compressive behaviour as the stress level increases. The more compressible (Sydney) specimen shows a response similar to the higher stress GC9-T1 even though the stress level ($\sigma'v$ = 220kPa) is similar to GC9-T3. The shear response of GC9-T6, which has similar compressibility to the Sydney specimen also shows the tendency for increasing brittleness with increasing stress level, however, this result is not considered reliable due to non-uniform deformation during shearing. This pattern of reducing dilation and increasing brittleness with stress level can explain why deeper failure surfaces develop.

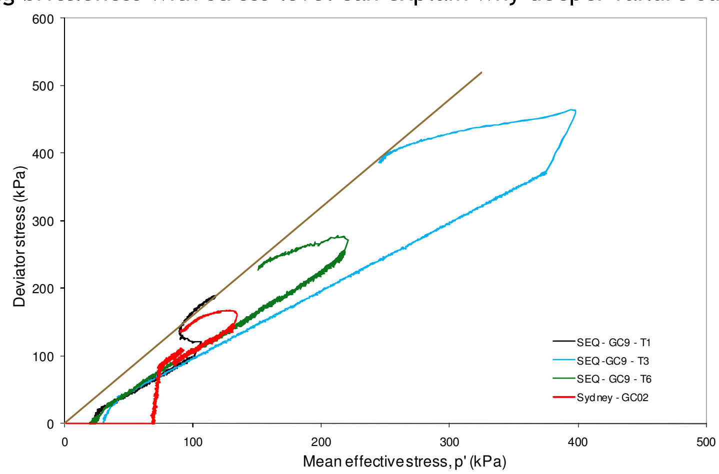

Figure 8: Stress-path curves from triaxial tests.

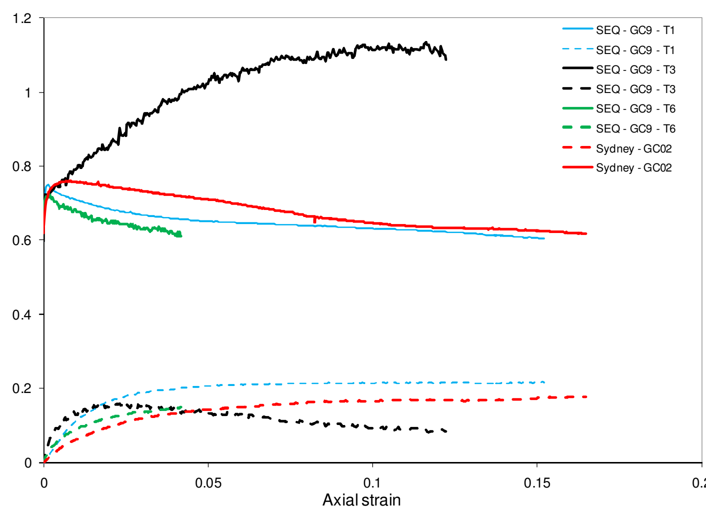

Figure 9: Normalised deviator stress and pore pressure responses from 1-D compressed specimens.

Case History (Continued)
Submarine Landslides on the South-Eastern Australian Margin

From Figure 9 it can be seen that all specimens approach a similar ultimate frictional resistance, which for the specimens shown ranged from 37^o to 40^o. Cyclic shear box tests showed no evidence of any lower residual frictional strength (Glenn *et al.*, 2008).

3. ANALYSIS

3.1 Landslide Initiation

Geomechanical modeling of three of the submarine landslides has been undertaken using the slope stability program GEO-SLOPE/W (2007) to examine the influence of cohesion, friction angle and slope geometry on the stability (Clarke *et al*, 2011). As the friction angle is around 40^o and the slopes are from 3^o to 6^o static analyses predict very high factors of safety. Analyses have also been conducted to investigate the effects of earthquake loading by including a factor for seismic accelerations in the standard pseudo-static limit-equilibrium calculations. Selecting an appropriate value for the seismic coefficient acting on the failure mass can be especially difficult (Seed and Martin 1966). A very crude investigation of seismic loading on the slopes indicates that lateral and vertical accelerations of 0.3 g (a_h = 0.3 g, a_v = -0.3 g), the upper limit of those used to investigate the stability of earth dams during earthquakes (Seed and Martin, 1966; Ozkan, 1998), would be sufficient to destabilise the slopes of the seafloor in the present study. While this approach has been widely used to assess the stability of submarine slides to earthquake events, its applicability to such large volumes of soil is questionable and the approach is of limited value in understanding the mechanisms leading to failure.

Puzrin *et al*. (2004) have argued that it is unlikely that a failure can develop over distances of several kilometres instantaneously, and that a progressive failure mechanism must be considered. On land progressive failures are often observed to result from oversteepening of the toe of a slope, where failure at the toe leads to a retrogressive failure that migrates upslope. This mechanism is also considered to be responsible for the large Storegga submarine slide. Puzrin *et al*. (2004) have suggested an alternative progressive failure mechanism that involves a weakened zone propagating down slope. The basis of the analysis can be explained by considering Figure 10. The starting point is that a zone of elevated pore pressures develop (Figure 10a), possibly owing to an earthquake, where the soil reaches a state of failure for which the mobilisable soil resistance is lower than the stresses at equilibrium from the weight of the overlying soil ($\tau_r < \tau_g$ Figure 10b). If the length of this failed zone is sufficient a global and catastrophic failure will occur. However, if the zone of failure is more limited the soil will tend to move downslope into the currently unfailed region. If the failure plane can propagate because the energy released is greater than that needed to progress the failure, then the shear plane can grow, and if conditions are unfavourable, it may continue to advance until it reaches a length where global failure results. For significant energy release to occur the ultimate resistance of the soil needs to be lower than that required to resist the gravitational stresses, and the soil needs to respond in a brittle manner. The triaxial test data shown above display the type of brittle behaviour that can potentially lead to this type of mechanism.

The analysis of Puzrin *et al*. (2004) suggests that failure begins upslope, but depending on the soil type and behaviour progressive failure may be limited or not occur and it is possible that the resulting length of the failure surface may be less than the critical value required for a catastrophic failure. There is some evidence for this from a number of head scarps present on the SE Australian margin where the soil below has not moved significantly. Glenn *et al*. (2008) suggest that these features represent the sites most likely to fail in the future. However, if the analysis of Puzrin *et al*. (2004) is correct, the soil movements make these sites less likely rather than more likely to lead to failure.

Case History (Continued)

Submarine Landslides on the South-Eastern Australian Margin

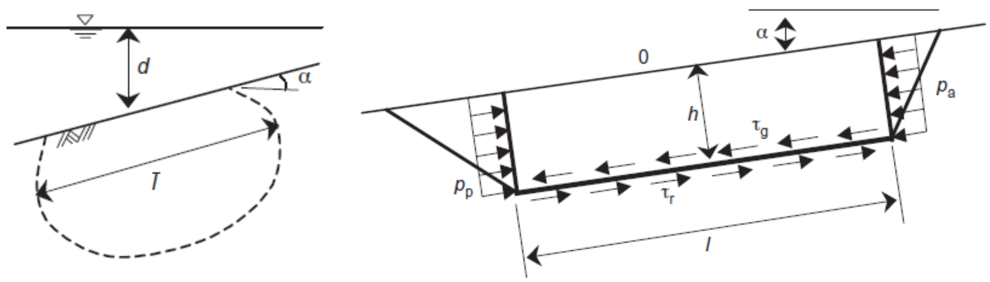

Figure 10: (a) Zone of elevated pore pressures, (b) Slope failure mechanism (after Puzrin *et al.*, 2004)

3.2 Tsunami generation

The viscous drag on the overlying ocean due to the movement of the slide block is responsible for the tsunami generation. Many studies have been conducted into tsunami generation and propagation, but only the work of Ward (2001) will be mentioned here. Ward (2001) presents results of tsunami generation and propagation based on classical tsunami theory and assuming linear wave theory. This theory uses a rigid seafloor overlain by an incompressible, homogeneous and non-viscous ocean subjected to a constant gravitational field. Figure 11 provides an indication of the size of the tsunami from analysis of a rectangular block of length L (km) and width W (km) sliding down an inclined plane for a water depth of 1000 m. The tsunami velocity is given by $\sqrt{(gh)}$ where h is the water depth, and in 1000 m of water this is 99 m/s. There are no reliable data on the speed of submarine slides, although turbidity currents from the 1929 Grand Banks slide travelled at 25 m/s, and based on travel distances of slide debris speeds of up to 80 m/s have been inferred for some large slides (Masson *et al.*, 2006). The fracture mechanics approach proposed by Puzrin *et al.* (2010) enables an estimate of the initial velocity to be determined and values of around 10 m/s were estimated for some reported slide geometries. For the largest slides on the SE Australian margin the water depth is around 1000 m, the maximum slide thickness is 200 m, and assuming a maximum velocity of 20 m/s a peak tsunami wave height of around 100 m can be estimated from Figure 11. As the maximum dimensions of the sliding blocks on the SE Australian margin are 20 km x 5 km some reduction to this height may be appropriate. Further increment in wave height will occur as any waves approach the coast.

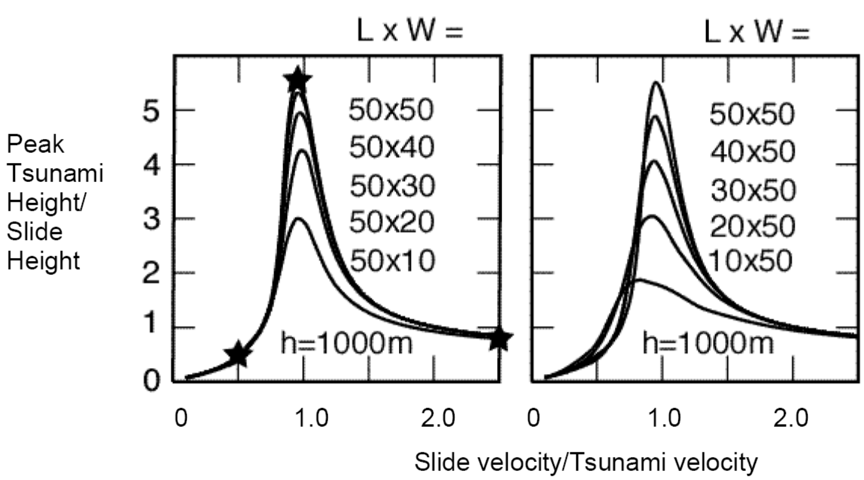

Figure 11: Effects of slide velocity and slide dimensions on peak tsunami height (after Ward, 2001).

Case History (Continued)

Submarine Landslides on the South-Eastern Australian Margin

4 SUMMARY

The paper has shown evidence of large scale mass wasting phenomena on the SE Australian continental slope, including the existence of many landslide features. Slides are evident all along the margin from Shoalhaven to the Sunshine Coast, and on slopes ranging from 1^o to 9^o. The soil properties of the upper sediments are similar along the margin and show no evidence of weak clay layers, although they do contain significant amounts of clay. The friction angles of the sediments are in the range of 37^o - 40^o, so that conventional soil mechanics would suggest the slopes have high factors of safety. However this is clearly not the case, as slope failures are widespread. Triaxial tests have indicated a significant increase in the brittleness of the shear response with stress level, and this is thought to be significant in explaining why the slides have thicknessess of 50 m to 200 m. The largest slope failures have a volume of 20 km^3 and have the potential to generate significant tsunami waves.

The dating of the slides suggests that the most recent failures occurred at the time when the sea level was at its minimum during the last glaciation. There are several reasons why the likelihood of slides should increase at these times, however the cause of the slides on the SE Australian margin is not well understood. While the likelihood of slides appears to be lower in inter-glacial periods there are examples of earthquake caused submarine slides that have occurred recently and the possibility that a large submarine slide could occur any day cannot be discounted.

5 ACKNOWLEDGEMENTS

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Case History (Continued)

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Case History

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

By Patrick K. Wong, Senior Principal, Coffey Geotechnics Pty Ltd, Sydney, Australia

1. Introduction

In 2006 to 2007, a reclamation was carried out on a coastal area in south-east Asia for the development of a container terminal which has a design elevation of RL+4.5m. The subsurface profile at the site comprised over 30m of soft to firm clay. The client implemented a program of surcharging with PVD for the container yard to limit the post-construction settlement to 300mm in 20 years, but let the 500m long wharf construction using a Design and Construct delivery mechanism. The wharf construction contract was awarded in 2007, and the successful contractor elected to use CDM in a 24m wide zone behind a piled wharf deck structure to provide the necessary stability for a dredging level which varied from RL-5m at the landward edge of the wharf deck to RL-15.7m at the seaward edge of the wharf.

The CDM zone immediately behind the wharf was constructed using overlapping 1.6m diameter columns to form interlocking 24m wide panels that run normal to the wharf alignment at a spacing of 3.6m centres, giving an area replacement ratio of 40%. The CDM columns were fully penetrating to RL-35m into very stiff to hard clay, and the CDM mix provided an average unconfined compressive strength of 1.3MPa (design strength of 1MPa). This zone is thus relatively stiff, and the post-construction settlement under the design loading of 40kPa was assessed to be 35mm. Based on monitoring results of the surcharged container yard, post-construction settlement in 20 years was estimated to be 315mm.

Therefore, the challenge was to design the transition zone to limit differential settlement to acceptable limits for drainage and pavement performance.

2. Differential settlement design criteria

The following design criteria were specified by the client with respect to limitations on differential settlement for an applied loading of 40kPa in the transition area:

- In 20 years, the minimum ground slope shall not be less than 0.7% in order to maintain adequate drainage within the container handling area.
- Within the transition zone, the differential settlement shall not be more than 0.3% change in grade from the general ground slope for satisfactory performance of the pavement.

Another challenge was that there were considerable uncertainties in post-construction settlement predictions particularly for the surcharged PVD area which is beyond the responsibility of the wharf construction contractor, but has direct impact on the differential settlement of the transition area.

3. Subsurface profile and soil properties

The soil profile at this site is relatively uniform, and comprised a thick soft to firm clay layer that exhibits linearly increasing stiffness and strength with depth. The soft to firm clay is underlain by a medium dense to dense sand followed by a deep sequence of very stiff to hard clays. The adopted parameters for the soft clay layer are summarised in Table 1.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Table 1 – Geotechnical Model Adopted

Layer	Elevation RL (m)	Thickness (m)	CR = $C_c/(1+e_o)$	CRR = $C_r/(1+e_o)$	$C_{\alpha\varepsilon}$	σ_{vo}' (kPa)	OCR	S_u (kPa)
1	+2 to +0	2	0.280	0.045	0.0112	13.22	4.9	11
2	+0 to -3	3	0.469	0.047	0.0188	28.28	3.5	15
3	-3 to -8	5	0.573	0.101	0.0229	43.18	1.9	19
4	-8 to -12	4	0.512	0.091	0.0205	66.69	1.6	25
5	-12 to -18	6	0.807	0.103	0.0323	98.17	1.7	33
6	-18 to -22	4	0.675	0.107	0.0270	122.68	1.8	49
7	-22 to -25	3	0.541	0.116	0.0217	139.16	1.6	56
8	-25 to -28	3	0.490	0.102	0.0196	156.52	1.6	63
9	-28 to -32	4	0.379	0.105	0.0152	174.47	1.5	72

where:
C_c = Compression Index
C_r = Recompression Index
e_o = Initial void ratio
$C_{\alpha\varepsilon}$ = Creep strain rate
σ_{vo}' = Initial effective vertical stress
S_u = Undrained shear strength
OCR = over-consolidation ratio

4. Adopted solution

The strategy adopted for the transition zone to meet the differential settlement design criteria was as follows:

- Surcharge the transition zone (5m surcharge height), which was carried out over a period of only 3 months due to time limitations. Only 0.65m of settlement was achieved in the surcharged transition area compared to about 3m in the surcharged PVD area.
- Provide an initial ground slope of 2.1% at the transition zone, sloping down from the edge of the surcharged PVD area to the edge of the wharf CDM area. This slope was chosen on the basis of the maximum slope at which the container handling over-head gantry will be able to operate. It is expected that this slope will reduce with time as the surcharged PVD area will settle more than the transition zone.
- Provide stepped CDM ground improvement in the 30m wide transition zone to provide a gradual increase of settlement towards the surcharged PVD area.

The CDM in the transition zone comprised twin 1.6m diameter columns at 4.8m lateral spacing and 3.5m longitudinal spacing, giving an area replacement ratio of 22.8%. An average of 8 rows of twin CDM columns was used across the transition area. The adopted strategy in the transition zone is illustrated in Figure 1 below.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

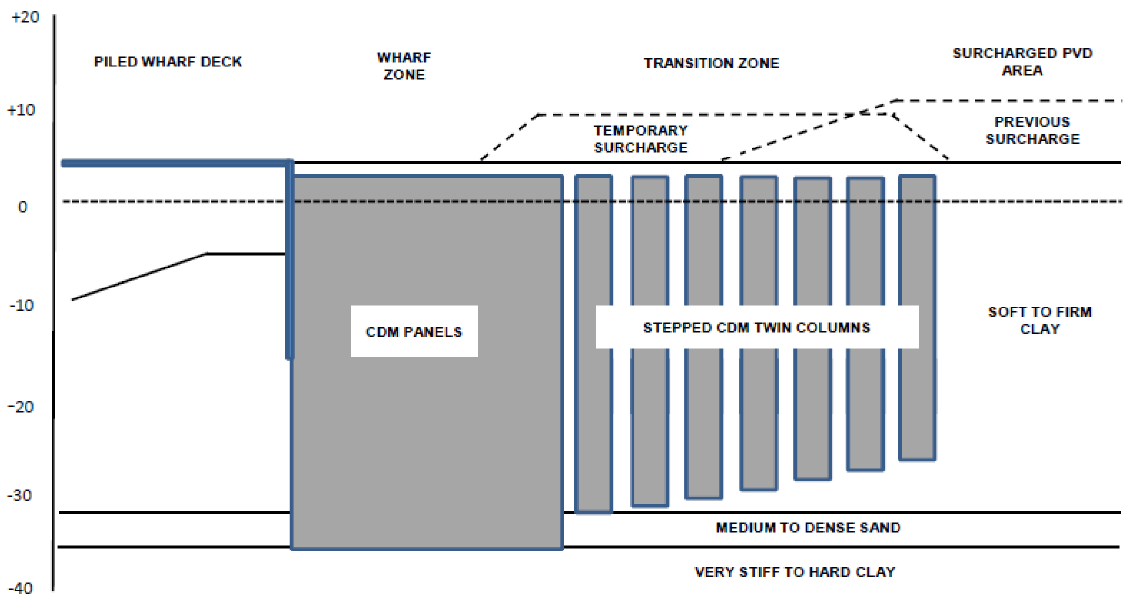

Figure 1 – Illustration of Ground Improvement Adopted in Transition Zone

5. Analysis methodology and results

The analysis method adopted for the design comprised primarily of simple one-dimensional consolidation analysis based on the method described in SGF (1997). The elastic modulus of the CDM column material was assessed to be 153MPa with a standard deviation of 28MPa based on 14 batches of tests (over 80 samples). The soil compressibility values given in Table 1 were converted to constrained modulus values based on initial and final stress levels, and OCR of the soil layers. An equivalent constrained modulus of the soil was then calculated in the CDM treated zone based on the area replacement ratio in accordance with SGF (1997). In the untreated zone below the toe of the stepped CDM columns, the soil compressibility is unchanged. Post-construction creep following preloading was assessed using the method described in Wong (2007).

After the design was approved by the client, numerical analyses were carried out using the commercially available finite element analysis software package PLAXIS. However, this paper will focus on the reliability assessment of the results rather than the analytical and numerical settlement analysis procedures.

By progressively lifting the toes of the twin CDM columns by 0.97m increments for each row moving landwards, the post-construction settlement in 20 years was estimated to range from about 60mm to 250mm, with increasing settlement towards the surcharged PVD area. With the initial ground slope set at 2.1%, the estimated settlement will reduce the initial slope to 1.4% in 20 years and this allows for uncertainty in predictions in meeting the minimum gradient of 0.7% for drainage requirements.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

The numerical analysis showed that because there is at least 2.5m of compacted fill over the soft clay, the differential settlement between CDM columns to be well within the design limit of 0.3% change from the general ground slope. Geogrid reinforcement in the fill was used to provide additional safety to even out settlement.

6. Reliability assessment

To assess the confidence level of the post-construction settlement estimate, a reliability assessment based on the procedure described by Duncan (2000) was carried out. The steps involved in the reliability assessment procedure is summarised briefly as follows:

(1) Assess the most likely values (MLV) for each parameter.

(2) Using the MLV for all parameters, calculate the most likely settlement estimate S_{MLV}.

(3) Assess the standard deviation (SD) of each parameter that involves uncertainty. In the absence of adequate statistical data, the standard deviation may be estimated in two ways (a) by using the three-sigma rule - estimating the highest conceivable value (HCV) and the lowest conceivable value (LCV) and estimating the standard deviation as (HCV – LCV)/6, and (b) by using published literature on the coefficient of variation (CV) and estimating the standard deviation as CV x MLV.

(4) Compute the settlement with each parameter increased by one SD and then decreased by one SD, while maintaining all other parameters at the MLV.

(5) Calculate the difference in settlement (ΔS_i) between (MLV + SD) and (MLV – SD) for each of the parameters, and use Taylor's series to calculate the combined standard deviation of S_{MLV} as follows:

$$SD_{MLV} = \sqrt{\left(\frac{\Delta S_1}{2}\right)^2 + \left(\frac{\Delta S_2}{2}\right)^2 + \left(\frac{\Delta S_3}{2}\right)^2 + \ldots\ldots\ldots\ldots} \qquad \text{Eq. [1]}$$

(6) Calculate the coefficient of variation of S_{MLV} as $CV_{MLV} = SD_{MLV}/ S_{MLV}$

(7) Assess the probability of the actual settlement being greater than SR x S_{MLV} where SR (settlement ratio) = actual settlement/S_{MLV} using a lognormal reliability index, β_{LN} as follows:

$$\beta_{LN} = \frac{\ln\{(SR)\sqrt{1+CV_{MLV}{}^2}\}}{\sqrt{\ln(1+CV_{MLV}{}^2)}} \qquad \text{Eq. [2]}$$

(8) Using the built-in function NORMDIST in Excel, the probability that the settlement ratio SR may be exceeded is {1 – NORMDIST(β_{LN})}

For the row of CDM columns on the side of the PVD area which are to be installed with their toes at about mid-depth of Layer 8, there are 16 parameters that involve uncertainties which will affect the estimated settlement. The adopted MLV, CV, SD, (MLV + SD) and (MLV – SD) for these parameters are presented in Table 2.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Table 2 – Parameters Adopted for Reliability Assessment

Description	Symbol	CV	SD	MLV	MLV+SD	MLD – SD
Applied stress	Δp	0.14	10	70.3	80.3	60.3
No of log time cycles for creep calculation	Ncreep	0.20	0.2	1	1.2	0.8
CDM treated zone	D'eq	0.20	7.3	36.5	43.8	29.2
	H(CDM)	0.02	0.5	28.5	29	28
Untreated bottom 1.5m thickness of Layer 8	CR	0.20	0.0980	0.4900	0.5880	0.3920
	CRR	0.20	0.0204	0.1020	0.1224	0.0816
	Cαε	0.20	0.0039	0.0196	0.0235	0.0157
	σvo'	0.05	8	160.1	168.1	152.1
	σp'	0.20	50.5	252.4	302.9	201.9
	H(clay)	0.00	0	1.5	1.5	1.5
Untreated clay layer 9	CR	0.20	0.0760	0.3790	0.4550	0.3030
	CRR	0.20	0.0210	0.1050	0.1260	0.0840
	Cαε	0.20	0.0030	0.0152	0.0182	0.0122
	σvo'	0.05	8.75	174.5	183.25	165.75
	σp'	0.20	52.8	264	316.8	211.2
	H(clay)	0.10	0.4	4	4.4	3.6

The adopted MLV and SD values were evaluated from an extensive set of testing results, together with back-analysis results of settlement from both the surcharge PVD area and the wharf CDM area.

The results from Step (4) of the reliability assessment procedure are presented in Table 3.

Table 3 – Computed Settlement from MLV + SD and MLV – SD for various Parameters

Description	Symbol	Settlement (m)			
		S(MLV + SD)	S(MLV - SD)	ΔS/2	$(\Delta S/2)^2$
Applied stress	Δp	0.260	0.235	0.012	0.000154
No of log time cycles for creep calculation	Ncreep	0.266	0.230	0.018	0.000325
CDM treated zone Untreated bottom 1.5m thickness of Layer 8	D'eq	0.238	0.261	0.011	0.000131
	H(CDM)	0.249	0.247	0.001	0.000001
	CR	0.250	0.246	0.002	0.000004
	CRR	0.251	0.244	0.004	0.000014
	Cαε	0.253	0.242	0.006	0.000035
	σvo'	0.254	0.240	0.007	0.000049
	σp'	0.243	0.304	0.031	0.000939
	H_c	0.248	0.248	0.000	0.000000
Untreated clay layer 9	CR	0.253	0.242	0.005	0.000030
	CRR	0.257	0.238	0.009	0.000086
	Cαε	0.260	0.235	0.012	0.000148
	σvo'	0.261	0.234	0.013	0.000173
	σp'	0.236	0.354	0.059	0.003475
	H_c	0.261	0.234	0.013	0.000180
			Standard Deviation		**0.076**

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

The combined SD on S_{MLV} calculated using Equation 2 is 0.076m (i.e. 76mm), giving a coefficient of variation CV_{MLV} of 76/250 = 0.3 (or 30%).

From Steps 7 and 8 of the reliability assessment procedure, the probability of exceeding a particular multiple of S_{MLV} (i.e. Settlement Ratio, SR) has been computed and shown in Figure 2 below:

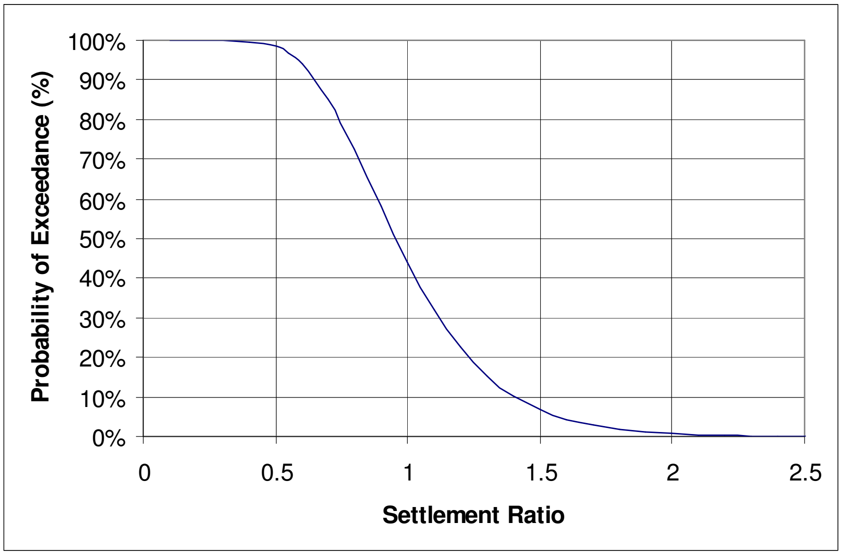

Figure 2 – Computed Probability Distribution of Settlement Estimate

For ground improvement design to limit settlement and differential settlement of civil infrastructures, a confidence level of 95% is generally considered to be a stringent design requirement. From Figure 2, it can be seen that there is only a 5% probability that the actual settlement would exceed the most likely estimate of 250mm by more than 1.6 times.

Assuming that the wharf CDM treated area would settle 60mm as estimated, and the settlement could be as much as 400mm (i.e. 1.6 x 250) in 20 years, the ground slope would reduce from the initial slope of 2.1% to about 1% which meets the specified minimum slope of 0.7% for drainage. Even if the wharf CDM treated area does not settle, a minimum slope of 0.7% in 20 years would be met.

7. Post-construction performance

Unfortunately, the author was unable to obtain any monitoring data from our client on this project post-construction. However, verbal information from our client is that the container terminal is performing satisfactorily to date. Plate 1 shows the pavement condition during operation of the container terminal.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Plate 1
Finished pavement surface during container handling

8. Conclusion

Different ground stiffness between the main container storage yard and the wharf area of this container terminal project was caused by different ground treatment adopted. The main container storage yard was treated using surcharge with PVD while the wharf area was treated using CDM with fully penetrating columns to RL-35m. This situation presented a significant challenge in the design of the transition zone between these two areas to meet the differential settlement criteria for serviceability of the container handling equipment, and surface drainage.

A 30m wide stepped CDM zone together with setting the initial ground slope upwards to the landside provided a satisfactory solution to the challenge. The use of the simple reliability assessment procedure described by Duncan (2000) provided a useful quantification of possible uncertainties, and provided confidence to the client that the adopted solution is sound, and has enough built-in safety to cater for potential uncertainties in material properties and design assumptions.

9. REFERENCES

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Case History
Geotechnical offshore site investigation and reclamation design at Port Kembla

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Tonilee Andrews, Port Kembla Port Corporation, Port Kembla

1. INTRODUCTION

Port Kembla is an active seaport situated on the north side of Red Point, approximately 90 km south of Sydney. As a result of the State Government's New South Wales Ports Growth Plan, a proportion of the shipping and cargo previously handled by Port Jackson has been relocated to Port Kembla. This development, combined with an on-going shortage of land within the Inner Harbour is understood to be the key incentive for the redevelopment of the Outer Harbour. In 2008 a major review of the development options for the Outer Harbour was performed, which considered contemporary commercial and trade related realities, and led to the proposed development being altered significantly from that of the previous development strategy. Prior to this, dredged spoil was deposited in the Outer Harbour within what was the footprint of the future reclamation. These activities resulted in a minimum of 460,000 m^3 of both imported slag and dredged spoil from the Inner Harbour being deposited in the Outer Harbour, over five disposal campaigns.

The PKPC Outer Harbour master plan proposes the reclamation of at least 42 hectares of additional port area over two stages of reclamation works, and the addition of 1770 m of new berth length. Stage 1 and 1A of the Outer Harbour development would create one additional bulk cargo berth and approximately 10 hectares of reclaimed land.

The overall Outer Harbour development has been divided into the following stages:

- Stage 1 and 1A - to create one additional bulk cargo berth and approximately 10 hectares of reclaimed land together with road connections (Phase 1).
- Stage 1B - the extension of the reclamation to the south, and eventually to the north, to incorporate the existing Port Kembla Gateway facility. This would then allow the extension of the bulk berth north and south to form a three berth facility (Phase 1).
- Stage 2A and 2B - to add a two berth container terminal and associated rail infrastructure (Phase 2).
- Stage 3 - to add two more container berths and associated reclamation, together with further development of associated rail and road infrastructure (Phase 2).

The PKPC Outer Harbour master plan showing these various stages are shown in Figure 1.

In February 2010, PKPC awarded SMEC the contract to undertake both Phase 1 and Phase 2 detailed geotechnical site investigation works, the associated detailed design of reclamation Stages 1 and 1A, and concept design options for Stage 1A berth.

This paper presents the findings of the offshore geotechnical site investigation and the design methodology and analysis results for the associated reclamation design of Stage 1 and 1A.

Case History
Geotechnical offshore site investigation and reclamation design at Port Kembla

2. Regional geology

The site is situated near the southern margin of the Sydney Basin. The 1:100,000 scale geological map of Wollongong-Port Hacking indicates that the site is directly underlain by Quaternary quartz and lithic fluvial sand, silt and clay. Immediately to the west of the site the Dapto Latite Member is indicated, comprising a melanocratic coarse grained and porphyritic latite. The Budgong Sandstone Formation is indicated approximately 3 km north-west of the site.

Bedrock at the site comprises the Budgong Sandstone Formation, derived from the lithification of a Permian marine deltaic sand. The Budgong Sandstone is the uppermost unit of the Shoalhaven Group, and outcrops along the coastal plain of Wollongong. The contact with the overlying Illawarra Coal Measures is conformable. It contains minor, interbedded, thin laminated siltstone, thin lenticular conglomerates and five tabular latite bodies. The sandstone is lithic to felspathic litharenite, and comprises mainly volcanic rock fragments and feldspar clasts (Bowman, 1971). Most of the Budgong Sandstone is planar bedded in laterally discontinuous units varying in thickness from several centimetres to 3 m. Bioturbation completely obliterates most bedding structures (Sherwin & Holmes, 1982).

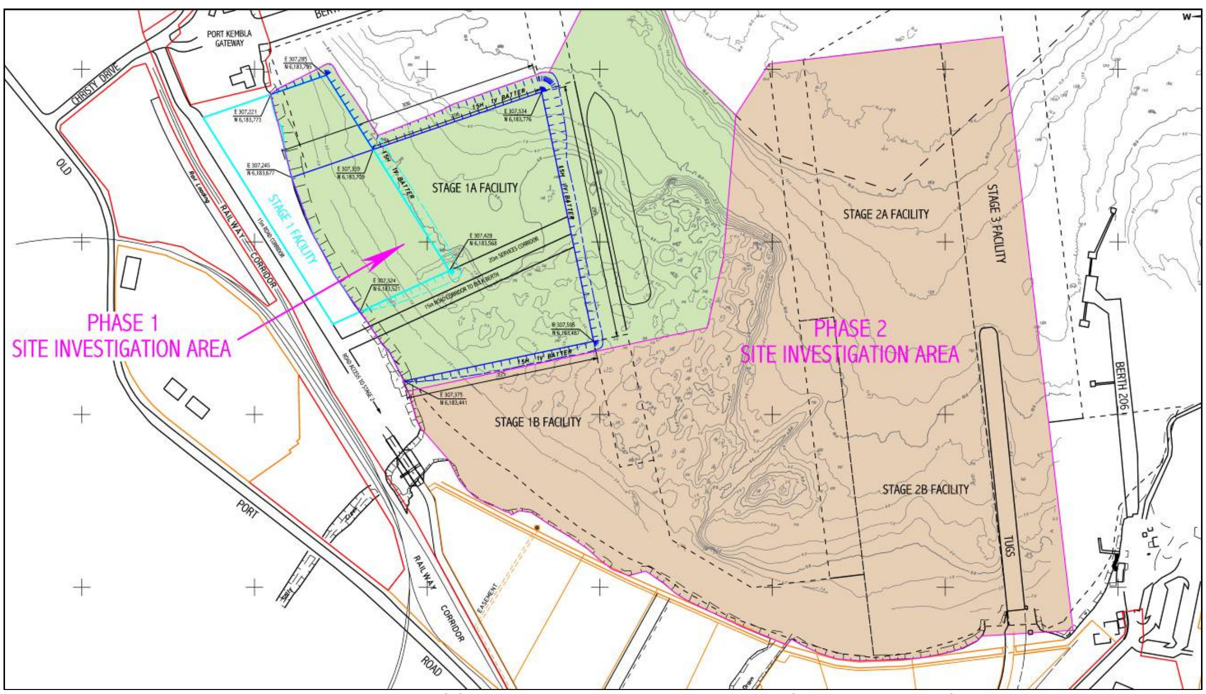

Figure 1: Port Kembla Port Corporation Outer Harbour master plan

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

The Dapto Latite Member is the most mafic of all the flows in the Gerringong Volcanic Facies. Petrologically, the Dapto Latite Member is basalt which varies in texture from aphanitic to porphyritic with a crystalline groundmass (Bowman, 1971). The Dapto Latite Member exhibits columnar jointing in some areas; it also contains partially or completely filled vesicles apparently elongated in stringers parallel to the flow direction. The Dapto Latite probably flowed into shallow water offshore, intruding soft sediments in part. Where the flows are in contact with the Budgong Sandstone, significant changes to weathering effects have not been reported.

Overlying the old eroded land surface are unconsolidated sediments. The clay, sand and gravel basal units are believed to be Quaternary alluvium, overlain by unconsolidated silts and clays, interpreted as modern marine and estuarine sediments.

3. Geotechnical offshore site investigation

As part of the development planning of the Outer Harbour, numerous geotechnical investigations had been undertaken. These include the drilling of 40 boreholes and vibrocores between 1977 and 2008.

A total of eighteen (18) boreholes (BHS101 to BHS118) were completed as part of the Phase 1 investigations, using a combination of washbore and bedrock core drilling methods for soil and rock respectively. Investigation locations were targeted to provide geotechnical data on the proposed areas of reclamation, dredging and construction. Borehole locations, the footprint of existing bunds together with nomenclature of bunds and reclamation areas are presented in Figure 2. For clarity and ease of reference, the Stage 1 and 1A containment bunds and reclamation have been divided into discrete sections and areas, and are also shown in Figure 2, as summarised below:

- Stage 1 containment bunds: B1 to B4
- Stage 1 reclamation areas: “General Area” and “Service and Road Corridor”
- Stage 1A containment bunds: B5 to B10
- Stage 1A reclamation areas: Areas R1 to R4

The recovery of undisturbed samples of the fill that were of sufficient size to permit triaxial and/or oedometer tests proved unsuccessful. This was due to a combination of particularly low shear strengths and relative densities allowing samples to ‘flow’ out of the U50/U63 tubes, and the presence of obstructions (such as slag) within the fill, which inhibited sample collection. The latter of these occurred twice in fill soils, resulting in damage to the sampling tube. Of the 58 SPT samples recovered from the investigation, five samples of fill were lost due to poor consolidation, and three samples lost due to obstructions (metal wire/slag).

4. Site geotechnical conditions and geotechnical interpretation

4.1 geotechnical conditions in Stage 1 Facility

The general area of the Stage 1 facility lies outside of the existing footprint of the perimeter bund for spoil disposal, and was subsequently not subject to filling. Marine deposits (approximately 1.0m thick, increasing to up to 3 m thick towards the north) were encountered and described as firm to stiff silty clay. Very loose to dense alluvial sands were also encountered throughout the area directly overlying the residual soils, with a typical thickness of between 1 m and 2 m.

Residual soils directly overlie the bedrock across the Stage 1 facility, and were typically described as a stiff to hard clay. The residual soils thicken in the southern area of Stage 1, where up to 3 m was encountered.

The 20 m wide service corridor and 15 m wide road corridor at the southern end of the facility encroach into the existing spoil disposal cell. Up to 1.5 m of dredged spoil is anticipated to be present within this area.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

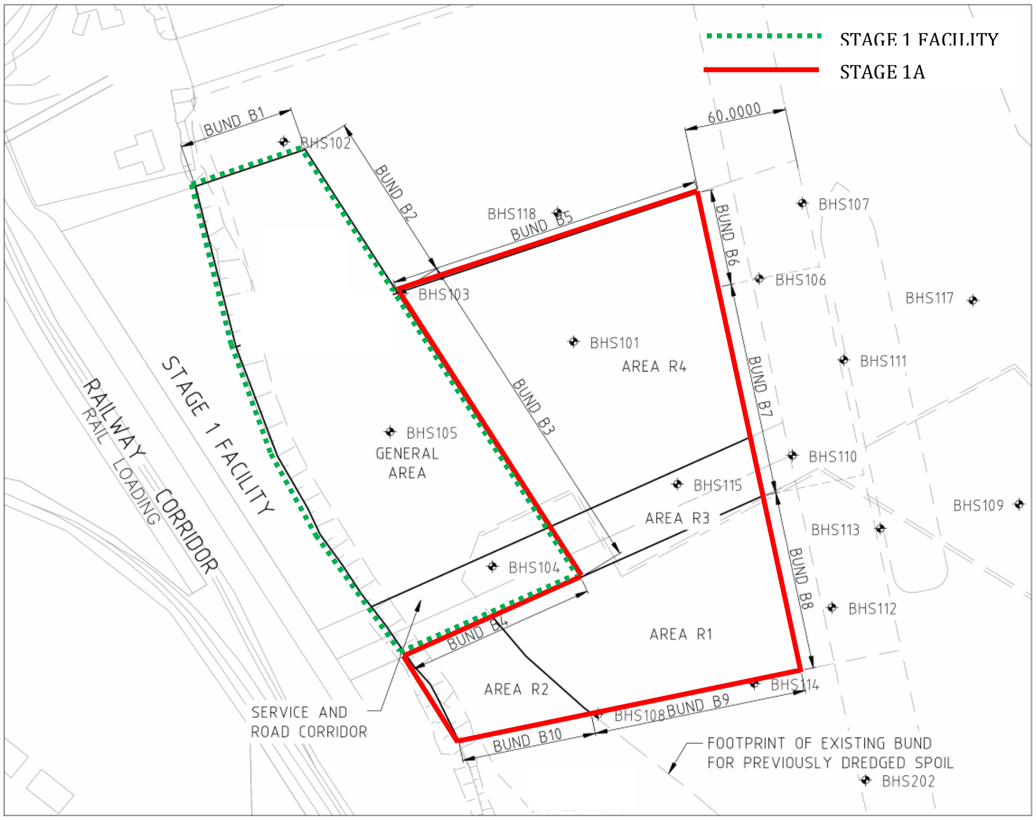

Figure 2: Geotechnical investigation location plan for Phase 1 investigations

4.2 geotechnical conditions in Stage 1A Facility

The underlying geotechnical conditions in the Stage 1A facility are significantly more complex than the Stage 1 area.

Unconsolidated dredged fill underlies the majority of the Stage 1A area and generally thickens towards the east and south-east of the area. The thickest sequences of dredged fill were encountered below the proposed eastern batter (Bunds B6 to B8), with no fill encountered at the northernmost edge of the proposed batter (Bund B5), increasing to 7.5 m of fill at the southernmost edge (Bunds B8 and B9). Fill was also absent from the far south-west corner (Bund B10). The nature of the fill is noted to alter towards the southern part of the area, as it changes from a granular material to a cohesive material.

Unconsolidated marine soils were encountered sporadically throughout the area, with thin (0.3 m to 0.5 m) laterally discontinuous occurrences. Alluvial soils were encountered throughout the area, with typical thicknesses of 1 m to 2 m. Thicker sequences of alluvium (up to 4.7 m) were noted to occur beneath the proposed southern batter in the vicinity of Salty Creek.

Residual soil comprises the base of the soil units, and is encountered directly overlying bedrock. The thickness of the residual soils varies between 0.4 m and 3.0 m, and is typically between 1 m to 2 m. The residual soils were effectively the only consolidated soil horizon in the Stage 1A area, and were typically stiff to hard clays.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

The depth below existing seabed level of the residual soils is noted to increase in a southerly direction. Towards the central and southern part of the Stage 1A area the residual soils were encountered at increasingly greater depths as the thickness of the overlying alluvial and dredged fill increases.

4.3 Geotechnical unitisation

Soil and rock units encountered concur well with published geological data, with six separate units being encountered during the investigation. The units encountered are presented in Table 1.

Table 1: Geotechnical unit descriptions

Unit	Description	Thickness (m)	Typical composition
1a 1b	Fill (cohesive) Fill (granular)	0.0 to 8.2	Poorly consolidated clay and sand fill mixed with variable minor fractions. Clay fill is very soft to firm, plasticity is variable. Sand fill is very loose to medium dense. Man made artifacts include charcoal, ash, slag gravels, possible coal-wash and metal wire.
2	Marine and estuarine sediment	0.0 to 1.0	Very soft to soft silty clay of variable plasticity. Only encountered as thin layers in BHS102, BHS106 and BHS111.
3a	Quaternary alluvium (cohesive)	0.0 to 3.4	Soft to firm clays were encountered within this unit. Shell fragments noted throughout.
3b	Quaternary alluvium (granular)	0.0 to 2.6	Typically very loose to medium dense sand with variable minor fractions. Shell fragments noted throughout.
4	Residual Soil	0.4 to 3.7	Typically very stiff to hard clays of low plasticity, with gravels of latite, sandstone and siltstone noted throughout. Sand and gravel units also encountered.
5	Dapto Latite Member	0.0 to > 1.0	Extremely weathered to highly weathered fine to coarse grained latite with medium to coarse gravels.
6	Budgong Sandstone Formation	>12.6	Extremely weathered becoming fresh sandstone and siltstone. Defect spacing and rock strength noted to increase markedly with depth.

4.4 Ggeotechnical design parameters

A suite of geotechnical design parameters was developed for the design of the reclamation. These parameters were derived from project specific in-situ and laboratory tests where available, and are considered to be representative of the properties of the material in its current condition. The geotechnical design parameters developed include:

- Bulk unit weight γ (kN/m^3)
- Undrained shear strength c_u (kPa)
- Effective cohesion c' (kPa) and effective friction angle ϕ' (degrees)
- Modified compression index $C_{c\varepsilon}$
- Modified recompression index $C_{r\varepsilon}$
- Modified secondary compression index $C_{\alpha\varepsilon}$
- Coefficient of consolidation C_v (m^2/year)

Drained elastic modulus E' (MPa)

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

A summary of the adopted geotechnical design parameters is given in Table 2.

Table 2: Geotechnical design parameters

Unit	Description	γ (kN/m^3)	c_u (kPa)	c' (kPa)	ϕ' (deg)	$C_{c\varepsilon}$	$C_{r\varepsilon}$	$C_{\alpha\varepsilon}$	C_v (m^2/yr)	E' (MPa)
1a	Fill (cohesive)	16	7.5	0	25	0.250	0.025	0.013	10	-
1b	Fill (granular)	16	-	0	30	-	-	-	-	7
2 / 3a	Marine estuarine Sediment Quaternary alluvium (cohesive)	17	10	0	22	0.250	0.025	0.013	5	-
3b	Quaternary alluvium (granular)	19	-	0	34	-	-	-	-	40
4	Residual Soil	19	150	5	28	0.100	0.010	0	50	-

5. Design options

As part of the design development, a number of different schemes were considered for the design of both the containment bund and the reclamation. The selection of the adopted solution and extent of ground improvement (if required) is highly dependent on the following factors:

- Capital cost
- Whole-of-life budgetary constraints
- Total and differential settlement criteria for the proposed use of the reclaimed land
- Construction program

5.1 Design options for containment bunds

The containment bunds could be placed directly on the seabed at locations where the geotechnical conditions are favourable, i.e. with little or no dredged fill and/or soft marine or alluvial soils. This applies to Bunds B1 to B3 of Stage 1, and Bunds B5 and B10 of Stage 1A.

At other locations, bund construction directly over dredged fill or unconsolidated soils would increase the risks of slope instability, thereby introducing an unacceptable element of risk to site operations in the short term, and in the long term over and beyond the project duration. This is particularly applicable for Bunds B6 to B8 of Stage 1A, where dredging would be undertaken in front of the bund for the Stage 1A berthing box, to RL-16.5m (PKD) well below the soil deposits and into the underlying rock mass.

Options that were considered to minimise the risk of slope instability include:

- Stabilising berms with/without high strength geotextile. This is applicable for Bunds B4 and B9/B10 that would be buried by future reclamations, and do not warrant complex or expensive treatment options.
- Dredging of soft sediment under the foundation of the bund. Subsequent disposal would be required prior to placement of bund materials. This is applicable for Bunds B6 to B8.
- Ground improvement of the soft materials in-situ, with various ground improvement techniques prior to bund construction. This is applicable for Bunds B6 to B8.

Preliminary slope stability analyses were undertaken for Bunds B6 to B8 for three options - dredged bund foundation, ground treatment with stone columns, and ground treatment with concrete injected columns. Comparative budget cost estimates were developed and it was found that ground treatment options (either stone columns or concrete injected columns) would cost approximately $10 million more than a dredged bund foundation option.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

Moreover, the use of ground treatment to improve stability would require strict quality control during construction, and adequate inspection and field testing to ensure that the design assumptions are met on site. This presented as an additional risk element to the design, is labour and time intensive, and any non-conformance would require additional design and remedial measures to be implemented during the construction phase.

Consequently, the detailed design for the containment bund involved:

- No treatment for Bunds B1 to B3 and B5.
- Use of stabilising berms and high strength geotextile (where required) for Bunds B4, B9 and B10.
- Dredging directly adjacent to the bund toe foundation along the eastern arm of Stage 1A (Bunds B6 to B8).

Removing dredged fill and soft sediments is a relatively lower risk option, as it does not rely on strict quality control during construction to ensure the installed ground improvement conform to design assumptions. To contain the dredged spoil, an additional containment bund would have to be constructed within the footprint of the future Stage 2A and 2B facility to contain the disposed material.

5.2 Design options for reclamation

Very soft to firm cohesive dredged fill and normally consolidated soft soils underlie the Stage 1A area south of the service corridor. Excessive consolidation settlement would occur if the reclamation fill and long-term design load are applied directly on these soft materials.

Consolidation settlement is the vertical displacement of the surface corresponding to the volume change due to the discharge of excess pore pressure set up by the increase in overburden load. In this instance, the overburden load equals the loading imposed by reclamation fill and long-term design load. The consolidation process continues until all the excess pore water pressure has completely dissipated.

Constructing buildings and infrastructure on under-consolidated ground may adversely impact their operation and performance, as excessive differential settlement may result in damage. Various ground improvement options have been considered to limit the post construction settlement. The possible options that could be adopted for the soft foundation materials include:

- Removal and replacement with reclamation fill.
- Preloading or surcharging to improve the in-situ ground after the reclamation. In this option, prefabricated vertical drains (PVD) can be installed into the soft materials to accelerate the discharge of excess pore pressure, if required.
- Installation of stone columns prior to the reclamation, followed by preloading and surcharging.
- Installation of rigid inclusions e.g. concrete injected columns after the reclamation has been completed to above the tidal zone (ie. RL +2.2 m)

The southern area of Stage 1A is underlain by soft marine sediments and cohesive dredged spoil deposited during past dredging campaigns. Hence, the reclamation may be prone to bearing capacity failure and excessive settlement. The prediction of post construction settlement for sites underlain by deep soft soil is associated with considerable uncertainties. Uncertainties in soil properties, including creep behaviour, and use of different design methods would alter the results.

The risk of the actual settlement exceeding the design value would be increased for non-rigid ground treatment options such as preloading, and reduced for structural support treatment options such as rigid inclusion techniques (ie. CICs).

A qualitative risk appraisal of the ground treatment methods is presented in Table 3.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

Table 3 – Risk appraisal of the proposed ground treatment for the reclamation

	Remove and Replace	Surcharge and Preload	Stone Column with Surcharge and Preload	Concrete Injected Columns (CICs)
Containment Bund failure during/post construction	Very Low	Medium / High	Low / Medium	Low
Required settlement period significantly longer than predicted	Very Low	Medium	Low	Very Low
Post construction settlement magnitude significantly larger than predicted	Very Low	Medium	Low	Very Low

Preliminary settlement analyses were undertaken for the last three options and comparative budget cost estimates were developed. The substantial volume of spoil generated by removal and replacement, the tight construction program and high costs for the stone column solution preclude these options from being adopted.

The detailed design hence included surcharge and preload for Areas R1 to R3, and the use of concrete injected columns for Area R4 where the thickest sequences of existing dredged spoil (Unit 1b) and marine/estuarine sediments and soft alluvial soils (Unit 2/3a) are present.

6 Containment bund and reclamation design

6.1 KEY DESIGN CRITERIA

6.1.1 Stability criteria

The following stability design criteria were adopted in the detailed design of the containment bunds. Two separate criteria were developed for permanent and temporary bunds.

Bunds B1, B2, B5 to B10 are considered permanent. They are exposed for an extended period of time before the reclamation is extended to the north (Bunds B1, B2 and B5) and to the south (Bunds B9 and B10) for the Stage 1B Facility, or before the wharf structure is constructed in front of Bunds B6 to B8. The minimum factors of safety adopted for design are summarised in Table 4 below.

Table 4: Summary of stability design criteria

Analysis case	Permanent bunds (B1, B2, B5 to B10)	Temporary bunds (B3, B4)
Short term	1.30	1.20
Long term	1.50	1.30
Seismic	1.10	1.10

6.1.2 Settlement criteria

Taking into consideration the intended future land usage, the design settlement criteria have been established for different areas, as shown in Table 5.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

Table 5: Summary of settlement design criteria

Stage	Area	Loading (kPa)	Total Post Construction Settlement Criteria
Stage 1	General area	20	50 mm PCS in 10 years
Stage 1	Service and road corridor	20	50 mm PCS in 10 years
Stage 1A	R1	50	50 mm PCS in 10 years
Stage 1A	R2	50	200 mm PCS in 10 years
Stage 1A	R3	20	50 mm PCS in 10 years
Stage 1A	R4	50	50 mm PCS in 10 years

6.2 Design methodology

The design of the reclamation was undertaken by considering the following:

- Global stability of containment bund. The analysis determined the slope stability of the reclamation and containment bund under short and long term loading, as well as during seismic events. The analysis was undertaken using the limit equilibrium software SLOPE/W, for both circular and non circular slip surfaces.
- Assessment of primary and post construction settlement of reclamation. The analysis has taken into account the preload and surcharge requirements, or the arrangement of ground treatment to satisfy the design criteria, in each reclamation area.

  The primary settlement and degree of consolidation was determined using the finite element program PLAXIS for the construction duration specified. When soft soil has been surcharged, the creep strain rate would reduce depending on the over-consolidation ratio achieved by the surcharge process. From the PLAXIS model, the degree of consolidation at surcharge removal was used to estimate the creep strain rate reduction (C_{α}'/C_{α}), which was then used to estimate the creep settlement, based on the method suggested by Stewart et al. (1994).
- Assessment of volumes of dredging, slag (a co-product of the iron making process) and interburden rock (latite breccia available from local quarries) required for bund construction, and volumes of slag required for reclamation.

6.3 Design summary

6.3.1 Stage 1

The geotechnical conditions within the footprint of the Stage 1 facility are relatively favourable and no foundation treatment is required for the construction of the bund. The northern and northeastern arms (Bunds B1 and B2) are "permanent" and will be constructed of interburden rock as Stage 1B would only be extended to the north after a minimum of 12 years. The southeastern (Bund B3) and southern (Bund B4) arms of the bund are only temporary and will be constructed of slag as they would be buried by the reclamation for Stage 1A, planned to be undertaken within the next two years.

No foundation treatment is required for the general area of the reclamation. However, the service and road corridor straddles the original footprint of the perimeter bund that contains the previously dredged spoil. To minimise any total and differential settlement, the foundation treatment consists of 2.3 m of surcharge (equivalent to 36 kPa) above RL +4.0 m for 3 months.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

6.3.2 Stage 1A

The geotechnical conditions under the northern arm (Bund B5) are relatively favourable and no foundation treatment is required for the construction of these bunds. The eastern arm (Bunds B6, B7 and B8) is the most critical section, as dredging would be undertaken in front of it for the Stage 1A berthing box down to RL -16.5 m. A dredged bund foundation is adopted for the entire length. The temporary trench would be filled with slag which would form the foundation of the bund up to RL -4.0 m. The bund would then be constructed of interburden rock directly on top of this slag foundation up to RL +2.2 m. For the southern arm (Bunds B9 and B10) the use of high strength geotextiles and stabilising berms are required to ensure slope stability.

For settlement control, the following are adopted for each reclamation area:

- Area R1
  This area is underlain by up to 7.5 m of dredged deposits and, in order to meet the settlement criteria, the foundation treatment for Area R1 consists of 0.45 m diameter CICs at 1.2 m centre to centre spacing in a triangular pattern.
- Area R2
  The foundation treatment for Area R2 consists of 5 m of surcharge (equivalent to 80 kPa) above RL +4.0 m for 3 months.
- Area R3
  The foundation material for Area R3 consists of with 2.3 m of surcharge (equivalent to 36 kPa) above RL +4.0 m for 3 months.
- Area R4
  The foundation treatment for Area R4 consists of 5 m of surcharge (equivalent to 80 kPa) above RL +4.0 m for 3 months.

Perspective view showing idealised profiles of the design, including the containment bunds and dredging for the bund foundation is shown in Figure 3 below.

This image was extracted from the three dimensional model developed for the reclamation which was utilised to develop the construction staging approach and materials volumes estimation required for the accurate pricing of the proposed works.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

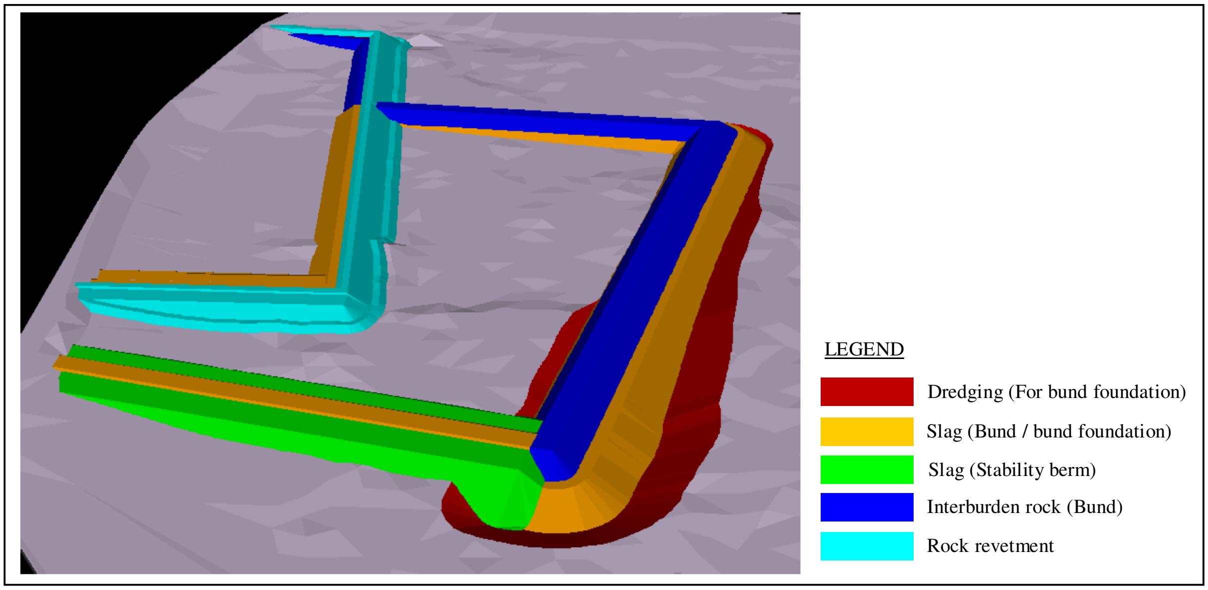

Figure 3: Perspective view showing completed Stage 1 and 1A bunds

7. Instrumentation and monitoring

Geotechnical instrumentation on the bund and reclamation area are required during and after construction in order to provide data that would enable:

- Confirmation of design assumptions e.g. the in-situ shear strength, the compressibility and the rate of consolidation. Due to the formation process of the cohesive dredged fill, it is expected that the properties would vary significantly across the site.
- Decision for preload/surcharge removal to be made by the Principal based on the performance during preloading. There are opportunities for early preload/surcharge removal if the rate of consolidation is faster than the predicted value. If necessary, contingency measures to be implemented in a timely manner.
- Recording of reclamation performance during construction to be kept for future reference.
- The geotechnical models representing the site conditions can be calibrated against the field measurements and performance. The calibrated geotechnical models can then be used to refine the post construction settlement predictions.

It was proposed that field monitoring be carried out regularly during bund construction and land reclamation in order to provide an early indication on any impending instability problems, and to monitor the performance of the preload and embankment founded in the soft soil areas. This included the installation of both settlement plates and settlement pins across the reclamation. The data from these instruments would be used to both confirm the design assumptions and also to establish the stability status of the bund and reclamation as they are being built.

The field monitoring would allow the risk of failure along the bund to be minimised and allow refinement of geotechnical models to update post construction settlement predictions. The following mitigating measures can be implemented in the event failure becomes imminent, without undue construction safety risk:

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

- Reducing the height of the reclamation
- Extending the period between lifts and wait for strength gains of the underlying soft cohesive soil

In the event the rate of settlement of preloaded embankment is slower than expected, the following measures could be adopted to rectify the situation:

- Leaving the preload/surcharge in place for an extended period of time
- Increasing the preload/surcharge height
- In extreme cases, contingency measures could include using ground inclusions to improve the strength of the ground.

8 Conforming and variation designs

8.1 Conforming design

The design detailed in Sections 6 and 7 above was the “conforming design”, which conformed to the original scope agreed with PKPC. It assumed that Stages 1 and 1A would be constructed in two stages, and a time lapse exists between the completion of Stage 1 and the commencement of construction of Stage 1A. Both Stages 1 and 1A (including all bunds and reclamation areas) would be constructed to their final configuration.

8.2 Variation design

In October 2010, following detailed pricing of the proposed scheme, and based on the direction from PKPC, the need for a lower cost solution was defined, leading to a revised concept of the containment bunds and reclamation for Stages 1 and 1A being developed. This revised concept, termed the “variation design”, adopted a high risk profile to the bund and reclamation design with lower performance requirements needing to be achieved.

In the “variation design”, the original Stage 1 and 1A would be constructed in one single stage, although the seaward bund of Stage 1A would be required to be located closer to the shore than in the original scope. The original Stage 1 area would be fully constructed, while the construction of the proposed bunds (B5 to B10) of the original Stage 1A would also be fully constructed. The remaining portions of the original Stage 1A may be constructed in separable portions.

No ground improvement or replacement was to be adopted for the variation design except in the areas which form the spine road and service corridor for the reclamation area. Early construction of pavements and services and hence controlled consolidation of this area are required. The ground improvement adopted in this area includes preloading the area with the proposed fill materials for a specified period of time to over-consolidate the soil, and then surcharging the ground with additional fill materials to achieve a reduction in post construction settlement. The variation bund design included the use of stability berms (and high strength geotextile where required) directly founded on the seabed, with no dredging of the existing spoil material. A typical section of the proposed design is given in Figure 4 below, which shows the slag bund, slag stabilising berms and high strength geotextiles and rock revetment which consists of primary and secondary rock armours.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

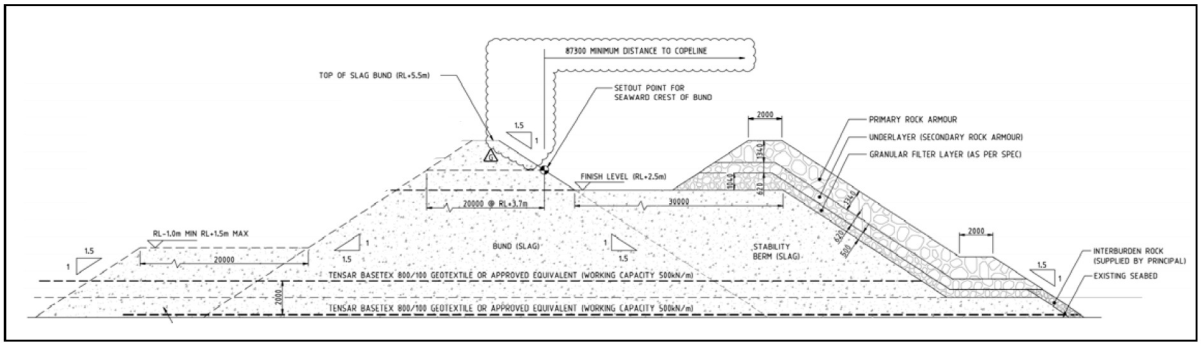

Figure 4: Typical section of bund for proposed variation design

For all other areas, the reclamation is allowed to settle, with no total or differential settlement criteria imposed on their performance. Notwithstanding this, at the southeastern corner of the reclamation where unconsolidated dredge material is the thickest, preloading of this area was recommended for a period of 6 months in order to allow the early stages of settlement to get underway and allow confirmation and future revision of the settlement performance for the area. The predictions for this area indicate that up to 1.2m of settlement may occur during this period, which would account for the majority of the predicted settlement, and would identify soft spots as a result of differential settlement.

This design has been put out to tender, a constructor selected, and construction is due to commence imminently.

8. CONCLUSIONS

The Outer Harbour of Port Kembla has been subjected to deposition of materials in the central and southeast sections of the works from five previous disposal campaigns, whereby dredged sediment from the Inner Harbour was relocated to the Outer Harbour. This paper has presented the methodology and results of geotechnical offshore site investigation at the Outer Harbour, and the associated detailed design of the reclamation.

Unconsolidated dredged fill underlie the majority of the works and generally thicken towards the east and southeast, where a maximum thickness of eight metres of dredged spoil was encountered. This presented a significant challenge to the design as the reclamation fill material would need to be founded on these soft deposits.

Phase 1 geotechnical design for the Outer Harbour development includes the design of containment bunds and land reclamation design associated with subsequent infilling with appropriate select fill material. Various design options were considered for both the bund and reclamation construction. Instrumentation and monitoring were proposed as part of the detailed design to confirm design assumptions and monitor the performance of the reclamation.

As the detailed design progressed, it was decided by PKPC that the conforming design which satisfies the original scope of works would not be constructed. Instead, a variation design consisting of the construction of all bunds, and reclamation areas without any intrusive ground improvement and an observational approach to the settlement performance was developed. No ground treatment was adopted, except for the areas which form the spine road and service corridor. This design has been adopted and will be implemented for construction commencing soon.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

Construction is commencing imminently and SMEC has been engaged by PKPC to act as the Principal's Representative to review the monitoring data obtained and provide design advice during construction.

9. REFERENCES

Bowman, H. N (1971) Geology of the Wollongong, Kiama and Robertson 1:50,000 Sheets, Geological Survey of New South Wales, Department of Mines.

Sherwin, L. and Holmes, G. G. (1982) Geology of the Wollongong and Port Hacking 1:100,000 Sheets, Geological Survey of New South Wales, Department of Mineral Resources.

Stewart, J.P., Lacy, H.S. & Ladd, C.C. (1994), Settlement of Large Mat on Deep Compressible Soil, Vertical and Horizontal Deformations of Foundations and Embankments, Proceedings of Settlement 94, ASCE Geotechnical Special Publication No.40, PP. 842-859.

Event Diary

ISSMGE EVENTS

Please refer to the specific conference website for full details and latest information.

2012

Second International Conference on Performance-Based Design in Earthquake Geotechnical Engineering
Date: 28 - 30 May 2012
Location: Conference Center , Taormina, Italy
Language: English
Organizer: ISSMGE TC-203
• Contact person: Dr. Claudio Soccodato
• Address: Associazione Geotecnica Italiana (AGI), viale dell'Università, 11
00185 Roma
Italy
• Phone: 39 064465569
• Fax: 39 0644361035
• E-mail: agiroma@iol.it
Website: www.associazionegeotecnica.it/novita

TC 211 International Symposium & Short Courses "Recent Research, Advances & Execution Aspects of GROUND IMPROVEMENT WORKS"
Date: 30 May - 1 June 2012
Location: IS: Crowne Plaza Brussels , Brussels, Belgium
Language: English
Organizer: TC 211 Ground Improvement
• Contact person: BBRI - Carine Godard
• Address: Avenue P. Holoffe 21
B-1342 Limelette
Belgium
• Phone: 32 2 655 77 11
• Fax: 32 2 653 07 29
• E-mail: carine.godard@bbri.be
Website: www.bbri.be/go/IS-GI-2012

12th Baltic Sea Geotechnical Conference
Date: 31 May - 2 June 2012
Location: Stadhalle (Town Hall) Rostock, Rostock, Germany
Language: English
Organizer: German Geotechnical Society
• Contact person: German Geotechnical Society
• Address: Gutenbergstr. 43
45128 Essen
Germany
• Phone: 49 201 78 27 23
• Fax: 49 201 78 27 43
• E-mail: service@dggt.de
Website: www.12bsgc.de

Shaking the Foundations of Geo-engineering Education (SFGE) 2012
Date: 4 - 6 July 2012
Location: NUI Galway , Galway, Ireland
Language: English
Organizer: ISSMGE
• Contact person: Dr. Bryan McCabe
• Address: Civil Engineering, National University of Ireland, Galway (NUI Galway)
Galway
Ireland
• Phone: 353 91 492021
• Fax: 353 91 494507
• E-mail: bryan.mccabe@nuigalway.ie
Website: www.sfge2012.com

11th ANZ 2012 Geomechanics Conference
Date: 15 - 18 July 2012
Location: Crown Promenade Hotel, Melbourne, Victoria, Australia
Language: English
Organizer: Leishman Associates
• Contact person: Leishman Associates
• Address: 113 Harrington Street
7000 Hobart
Tasmania
Australia
• Phone: 61 36234 7844
• Fax: 61 6234 5958
• E-mail: nicole@leishman-associates.com.au
Website: www.anz2012.com.au

Event Diary (continued)

6ICSE - 6th International Conference on Scour and Erosion
Date: 28 - 31 August 2012
Location: Ecole des Arts et Métiers, Paris, France
Language:
Organizer:
• Contact person: contact@icse6-2012.com
Website: www.icse-6.com

2nd International Conference on Transportation Geotechnics
Date: 10 - 12 September 2012
Location: Hokkaido University, Sapporo, Hokkaido, Japan
Language: English
Organizer: ISSMGE (TC202) and JGS
• Contact person: Dr. Tatsuya Ishikawa
• Address: Faculty of Engineering, Hokkaido University Kita 13, Nishi 8, Kita-ku
060-8628 Sapporo
Hokkaido
Japan
• Phone: 81-706-6202
• Fax: 81-706-6202
• E-mail: tc3conference@eng.hokudai.ac.jp
Website: congress.coop.hokudai.ac.jp/tc3conference/index.html

7th International Conference in Offshore Site Investigation and Geotechnics: Integrated Geotechnologies, Present and Future (12-14 September)
Date: 12 - 14 September 2012
Location: Royal Geographical Society , London, United Kingdom
Language: English
Organizer: TC209, SUT - OSIG
• Contact person: Peter Allan
• Address: Geomarine Ltd, A2 Grainger Prestwick Park
NE20 9SJ NEWCASTLE UPON TYNE
England
• Phone: 44 (0) 191 4537900
• E-mail: peter.allan@geomarine.co.uk; zenon@tamu.edu

The Seventh Asian Young Geotechnical Engineers Conference (7AYGEC)
Date: 12 - 14 September 2012
Location: The University of Tokushima , Tokushima, Tokushima, Japan
Language: English
Organizer: Japanese Geotechnical Society
• Contact person: Prof. Ryosuke Uzuoka
• Address: Dept. of Civil and Environmental Engineering, The University of Tokushima
2-1 Minamijyousanjima-cho
770-8506 Tokushima
Tokushima
JAPAN
• Phone: 81-88-656-7345
• E-mail: uzuoka@ce.tokushima-u.ac.jp
Website: sites.google.com/site/7aygec/

ISC'4 - 4th International Conference on Geotechnical and Geophysical Site Characterization
Date: 18 - 21 September 2012
Location: Porto de Galinhas, Pernambuco, Brazil
Language:
Organizer: TC102
• Contact person: Executive Secretary
• Address: Rua Ernesto de Paula Santos 1368, salas 603/604
Boa Viagem; Recife - PE CEP: 51021-330
Brazil
• E-mail: isc-4@factos.com.br
Website: www.isc-4.com/index.php

Event Diary (continued)

International Conference on Ground Improvement and Ground Control: Transport Infrastructure Development and Natural Hazards Mitigation
Date: 30 October - 2 November 2012
Location: University of Wollongong, Wollongong, New South Wales, Australia
Language: English
• Organizer: The Centre for Geomechanics and Railway Engineering, University of Wollongong, Australia, and the Australian Geomechanics Society (AGS)
. Contact person: Dr. Jayan Vinod
. Address: Centre for Geomechanics and Railway Engineering, Faculty of Engineering,
University of Wollongong,
2522 Wollongong,
New South Wales,
Australia.
. Phone: 61 02 4221 4089
. Fax: 61 02 4221 3238
. E-mail: icgi_2012@uow.edu.au
. Website: www.icgiwollongong.com
. Deadline for Abstract submission: 1 July 2011

2013

First Pan-American Conference on Unsaturated Soils (Pam-Am UNSAT 2013)
Date: 20 - 22 February 2013
Location: Convention Center, Cartagena de Indias, Colombia
Language: English
Organizer: UniAndes, UniNorte, Unal, Col
• Contact person: Diana Bolena Sánchez Melo
• Address: Carrera 1 Este No. 19A-40
Edificio Mario Laserna Piso 6
Departamento de Ingenieria Civil & Ambiental
Bogotá
Colombia
• Phone: 571 3324312
• Fax: 571 3324313
• E-mail: panamunsat2013@uniandes.edu.co
Website: panamunsat2013.uniandes.edu.co

18th International Conference for Soil Mechanics and Geotechnical Engineering
Date: 1 - 5 September 2013
Location: Paris International Conf. Ctr , Paris, France
Language:
Organizer:
• Contact person: Violaine Gauthier
• Address: Le Public Système,
38, rue Anatole France –
92594 Levallois-Perret Cedex
France
• Phone: 33 1 70 94 65 04
• E-mail: vgauthier@lepublicsysteme.fr
Website: www.issmge2013.org/

2014

8th European Conference on Numerical Methods in Geotechnical Engineering (NUMGE14)
Date: 18 - 20 June 2014
Location: Delft University of Technology, Delft, Netherlands, The
Language: English
Organizer: Prof. Michael Hicks
• Contact person: Mrs. Hannie Zwiers
• Address: Delft University of Technology, Faculty of Civil Engineering & Geosciences
Stevinweg 1
2628 CN Delft
The Netherlands
• Phone: +31 15 2788100
• E-mail: info@numge2014.org
Website: www.numge2014.org

Event Diary (continued)

NON-ISSMGE SPONSORED EVENTS

2011

Young Geotechnical Engineers Conference 2011 - South Africa
Date: 31 October - 2 November 2011
Location: Berg and Dal Conference Centre , Kruger National Park, Limpopo, South Africa
Language: English
Organizer: SAICE Geotechnical division
• Contact person: RCA Conference organisers - Yolandé Oosthuizen
• Phone: 27117288173
• E-mail: register@rca.co.za

International Conference on Advances in Geotechnical Engineering (ICAGE 2011)
Date: 7 - 9 November 2011
Location: Burswood Entertainment Complex , Perth, Western Australia, Australia
Language: English
Organizer: Curtin University
• Contact person: EEC W Pty Ltd, Australia
• Phone: 61-8-9389 1488
• Fax: 61-8-9389 1499
• E-mail: info@eecw.com.au
Website: www.icage2011.com.au

5th Asia-Pacific Conference on Unsaturated Soils
Date: 14 - 16 November 2011
Location: Pattaya , Pattaya, Thailand
Language: English
Organizer: Thai Geotechnical Society, KU
• Contact person: Apiniti Jotisankasa
• Address: Department of Civil Engineering, Kasetsart University
10900 Jatujak
Bangkok
Thailand
• Phone: 66819043060
• Fax: 6625792265
• E-mail: fengatj@ku.ac.th
Website: www.unsat.eng.ku.ac.th

Segunda Conferencia Ecuatoriana de Ingeniería Geotécnica y Ambiental para Ingenieros Jóvenes y Estudiantes (SCEIGA)
Date: 16 - 18 November 2011
Location: Universidad de Guayaquil , Guayaquil, Guayas, Ecuador
Language: Español
Organizer: SEMSIR
• Contact person: Maria Jose Avecillas Andrade
• Address: Laboratorio Ruffilli – Universidad de Guayaquil,
Av. Kennedy.
9176 Guayaquil
Guayas
Ecuador
• Phone: 59384862808
• Fax: 59342286290
• E-mail: aniversariosemsir50@gmail.com
Website: semsir.blogspot.com

GEOMAT 2011-MIE, JAPAN
Date: 21 - 23 November 2011
Language: English
Organizer: Glorious International GEOMAT
• Contact person: Dr. Zakaria Hossain
• Address: Assoc. Prof., Graduate School of Bioresources,
Mie University
514-8507 Tsu
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Japan
• Phone: 81592319578
• Fax: 81592319591
• E-mail: zakaria@bio.mie-u.ac.jp
Website: gipremi.webs.com/

Geotechnical Engineering Conferences of Torino (XXIII Edition) / Conferenze di Geotecnica di Torino (XXIII CICLO)
Date: 23 - 24 November 2011
Location: Politecnico di Torino , Torino, Italy
Language: Italian / English
Organizer: Politecnico di Torino
• Contact person: AXEA Conferences and Events
• Address: Via Caboto 44
10129 Torino
Italy
• Phone: 39011591871
• Fax: 39011590833
• E-mail: info@cgttorino.org
Website: www.cgttorino.org/

Event Diary (continued)

International Symposium on Sustainable Geosynthetics & Green Technology for Climate Change (SGCC2011)
Date: 7 - 8 December 2011
Location: Grand Centara Convention Hotel , Bangkok, Thailand
Language: English
Organizer: ACSIG, SEAGS
• Contact person: SGCC2011 Secretariat
• Address: c/o Asian Center for Soil Improvement and Geosynthetics (ACSIG); GTE/SET,
Asian Insitute of Technology
PO Box 4,
Klong Luang,
Pathumthani 12120
Thailand
• Phone: 66-2 524 5523
• Fax: 66-2 524 6050
• E-mail: climatechange@ait.ac.th or igs-thailand@ait.ac.th
Website: www.set.ait.ac.th/acsig/sgcc2011/home.htm

2012

4th International Conference on Grouting and Deep Mixing
Date: 15 - 18 February 2012
Location: Marriott New Orleans , New Orleans, LA, United States
Language: English
Organizer: ICOG and DFI
• Contact person: Theresa Rappaport
• Address: DFI; 326 Lafayette Avenue
07506 Hawthorne
NJ
USA
• Phone: 9734234030
• Fax: 9734234031
• E-mail: trappaport@dfi.org
Website: www.grout2012.org

Geo-Congress 2012
Date: 22 - 25 March 2012
Location: Oakland, California, United States
Language: English
Organizer: Geo-Institute of ASCE
• Contact person: Rob Schweinfurth
• Address: 1801 Alexander Bell Drive
Reston, VA 20191
United States
• Phone: 1.703.295.6015
• E-mail: rschweinfurth@asce.org
Website: www.geocongress2012.org

NGM 2012. 16th Nordic Geotechnical Meeting
Date: 9 - 12 May 2012
Location: Tivoli Congress Center, Copenhagen, Denmark
Language: English
Organizer: Danish Geotechnical Society
. Contact person: Morten Jorgensen
. Address: Sortemosevej 2
DK-3450 Allerod
Copenhagen
Denmark
. Phone: +45 4810 4207 ; +45 4810 4207
. Fax: +45 4810 4300
. E-mail: moj@niras.dk
Website: www.ngm2012.dk

11th International & 2nd North American Symposium on Landslides
Date: 3 - 8 June 2012
Location: Fairmont Banff Springs Hotel , Banff, Alberta, Canada
Language:
Organizer: CGS, AEG, JTC1
• Contact person: Wayne Gibson, P.Eng. Conference Manager
• Address: c/o Gibson Group Association Management,
8828 Pigott Road,
V7A 2C4 Richmond
BC
Canada
• Phone: 1 (604) 241-1297
• Fax: 1 (604) 241-1399
• E-mail: info@isl-nasl2012.ca
Website: www.isl-nasl2012.ca/index.php?lang=en

Event Diary (continued)

34th International Geological Congress (34th IGC)
Date: 5 - 10 August 2012
Location: Convention and Exhibition Ctr , Brisbane, Queensland, Australia
Language: English
Organizer: IUGS
• Contact person: For full contact details see - http://www.34igc.org/congress-manager.php
• Address: 34th IGC, PO Box 177
Redhill
Queensland 4059
Australia
• Phone: 61 7 3368 2644
• Fax: 61 7 3369 3731
• E-mail: info@34igc.org
Website: www.34igc.org/index.php

XXI Congreso Argentino de Mecánica de Suelos e Ingeniería Geotécnica (CAMSIG XXI)
Date: 12 - 14 September 2012
Location: Salón Terrazas del Parana, Rosario, Santa Fe, Argentina
Language: Spanish
Organizer: Soc Argentina Ing Geotecnica
• Contact person: Ing Virginia Sosa
• Address: Boulevard Oroño 1572 Planta Baja
2000 Rosario
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Argentina
• E-mail: secretaria@camsig2012.com.ar
Website: camsig2012.com.ar

IS-Kanazawa 2012, The 9th International Conference on Testing and Design Methods for Deep Foundations
Date: 18 - 20 September 2012
Location: Kanazawa Bunka Hall , Kanazawa, Ishikawa, Japan
Language: English
Organizer: Japanese Geotechnical Society
: • Contact person: Associate Prof. Shun-ichi Kobayashi
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920-1192 Kanazawa
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Japan
• E-mail: office@is-kanazawa2012.jp
Website: is-kanazawa2012.jp

GA2012 - Geosynthetics Asia 2012 - 5th Asian Regional Conference on Geosynthetics
Date: 10 - 14 December 2012
Location: Centara Grand, Bangkok Conv Ct , Bangkok, Thailand
Language: English
Organizer: IGS-Thailand
• Contact person: GA2012 Secretariat

• Phone: +66-2-524-5523

• Fax: +66-2-524-6050
• E-mail: igs-thailand@ait.ac.th or acsig@ait.ac.th
Website: www.set.ait.ac.th/acsig/GA2012/

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