SIMSG ISSMGE

ISSMGE Bulletin

Volume 4, Issue 2
June 2010

International Society for Soil Mechanics and Geotechnical Engineering

www.issmge.org

INSIDE THIS ISSUE

Message to ISSMGE from the President of the Geosynthetics Engineering and the International Geosynthetics Society (IGS)

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Dear Colleagues in the ISSMGE

It is a privilege for us, the current President and Immediate Past-President of the International Geosynthetics Society (IGS), to jointly share with you our thoughts about the many opportunities for collaboration between the ISSMGE and the IGS.

While man-made, geosynthetics belong to the family of “geomaterials.” They are defined as planar products manufactured from polymeric materials, which are used with soil, rock or other geotechnical engineering related material as an integral part of a man-made project, structure, or system. In fact, the advent of geosynthetics has augmented significantly the range of mechanical and hydraulic properties that engineers can now adopt in the design of geotechnical systems. The variety of geomaterials is huge, as illustrated in Figure 1, obtained from a State-of-the-Art Lecture presented in the 17ICSMGE in Alexandria. Note that the geomaterials shown in the last row of this figure illustrates geosynthetic materials. As in the case of soil and rock, the properties of geosynthetics should be properly characterized. It turns out that the characterization of geosynthetic materials has significant similarities with the characterization of soil and rock.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

Characterization of a particular soil formation may involve assigning quantifiable measures to describe their composition, gradation, particle shape, angularity, mineralogy, initial void ratio, density, packing arrangement, microstructure, fissuring, and/or degree of cementation. In the case of geosynthetics, additional properties that may require quantification include their tensile strength, stiffness, transmissivity, permittivity, creep rate, interface shear strength, and pullout resistance, to name a few.

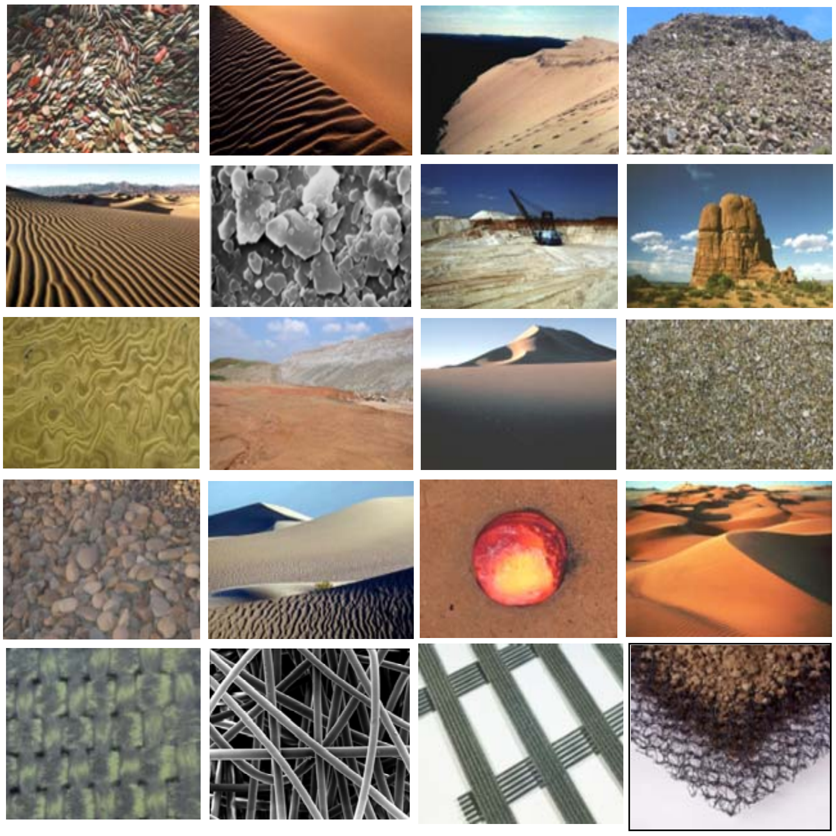


Figure 1. View of the diversity of geomaterials (Mayne et al. 2009)

The IGS is one of the learned international technical societies whose focus is closely related to Geotechnical Engineering, while also having close association to other disciplines such as Polymer Engineering. Building on the summary of membership compiled by Prof. Hudson in his recent article in this ISSMGE Bulletin (Hudson 2010). Table 1 provides a comparison of the membership of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), the International Society of Rock Mechanics (ISRM) and the International Geosynthetics Society (IGS).

Table 1. Membership of the ISSMGE, ISRM, and IGS

	ISSMGE	ISRM	IGS
Number of National Groups or Chapters	86	48	31
Number of Individual Members	18,323	5,992	2,263
Number of Corporate Members	21	125	141
Number of Technical Committees	24	9	3

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

While the number of individual members of the IGS is an order of magnitude smaller than that of ISSMGE and around half that of the ISRM, the IGS counts with a healthy corporate membership that exceeds the number of corporate members of our sister geo-societies. The presence of the IGS around the globe is increasing, and good evidence of such increase is the creation of new IGS chapters. Figure 2 shows the chronology of the formation of IGS chapters since the founding of the IGS in 1983. As shown in the figure, the number of chapters has been increasing at a remarkably high (and reasonably steady) rate of over one chapter per year (1.3 chapters per year, to be more precise). The IGS is now represented with chapters in all the continents. In addition to its chapters, which have traditionally conducted most of our technical activities, the IGS has initiated the implementation of Technical Committees (TCs). While its number is currently small (TCs on Soil Reinforcement, Barrier Systems, and Filtration), they are expected to grow rapidly and to provide significant opportunities for collaboration with the ISSMGE and other sister societies.

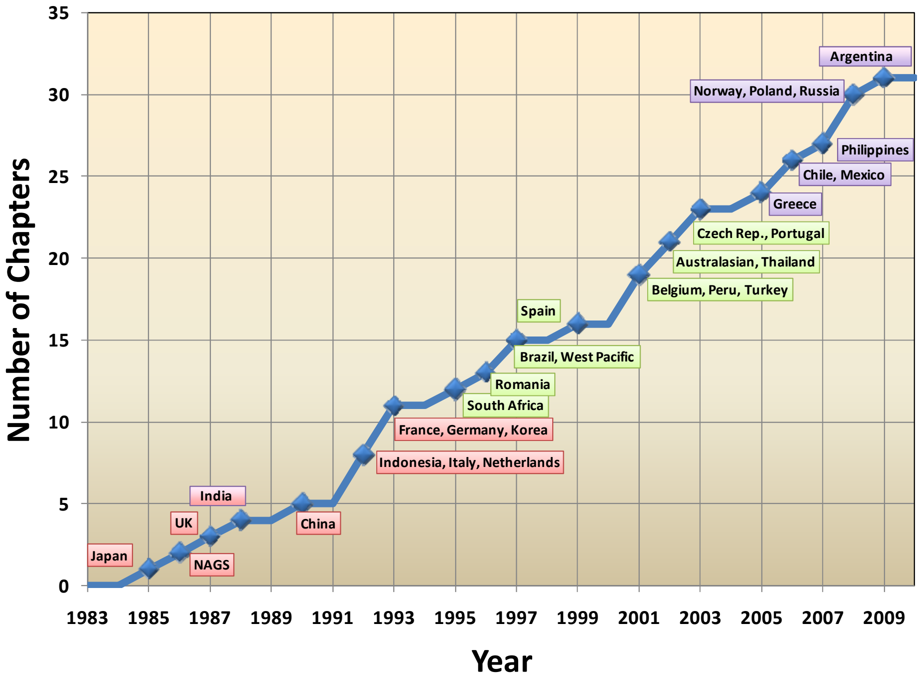


Figure 2. Chronology of the formation of IGS chapters

Following on this brief introduction, this article provides some basic information about geosynthetics, discusses the common knowledge base shared by ISSMGE and the IGS, provides an overview of the history of the IGS, and elaborates on the opportunities for collaboration between our Societies.

<u>Geosynthetics and Geosynthetics Engineering</u>

Geosynthetics have been used increasingly in Geotechnical and Environmental Engineering practice. We can say that geosynthetics is among the most relevant innovations in the field of Geotechnical and Environmental Engineering in the last half century. While one of the youngest disciplines, geosynthetics engineering has become a critical discipline in the field of modern Civil Engineering. This is because the use of geosynthetics is expected to continue to increase, and most probably at a fast rate. For example, their use has been identified as having the ability to reduce the total amount of carbon dioxides emitted in civil engineering construction project (e.g. by constructing geosynthetic-reinforced retaining walls rather than conventional reinforced concrete retaining walls). Geosynthetics are being increasingly used in geoenvironmental applications (e.g. in the design of waste containment facilities).

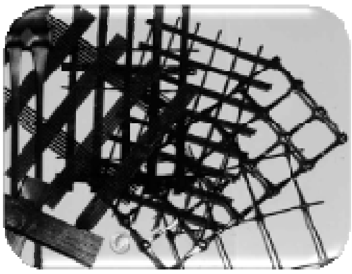
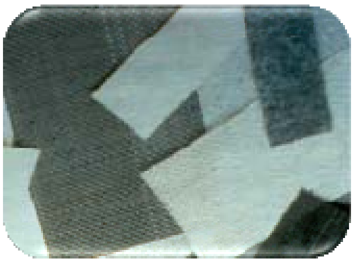
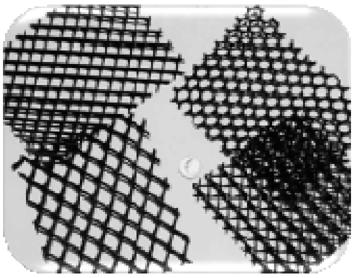
Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

They are also used as effective measures to mitigate natural disasters, as currently highlighted in the recent “Giroud Lecture” (the prestigious named lecture of our Society), delivered by Prof. Brandl (2010), which illustrates the use of geosynthetics in structures designed to sustain extreme loads induced by events such as earthquakes and flooding. According to Giroud (2008), it is justified to refer to geosynthetics as a full discipline, because, unlike other innovations in geotechnical engineering, geosynthetics have pervaded most branches of the geotechnical practice. The relevance of geosynthetics has increased rapidly due not only to their use in well-established applications but mainly to the increasing number of new applications involving their use.

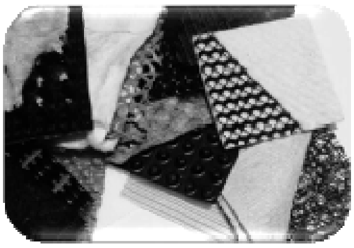
In order to fulfill design needs of geotechnical-, environmental-, and hydraulic-related systems, the geosynthetic industry has developed a number of products to achieve multiple functions. This includes the functions of separation, reinforcement, filtration, drainage, hydraulic barrier, and protection. The types of geosynthetics include, but are not limited to, those described in Table 2.

Table 2. Some of the most common types of geosynthetics

Geosynthetic type		Description
Geogrids		They provide reinforcement (mechanical stabilization) by allowing the development of tensile stresses within a soil mass. Projects include reinforced steep slopes, retaining walls, pavements and foundations. The tensile stresses induced within a soil mass help in a similar way to the tensile stresses induced in a concrete element by steel reinforcement. Geogrids have also been used to stabilize projects in landfill and mining facilities.
Geotextiles		They constitute fabric-like products, being grouped into nonwoven and woven geotextiles. They are commonly used in filtration applications for hydraulic systems, pavements and landfills. They are also used as cushion to protect geomembranes from puncture. Indeed, geotextiles can also be used in most of the geosynthetic functions, including in-plane drainage and reinforcement.
Geonets		They involve unitized sets of parallel ribs positioned in layers such that liquid can be transmitted within their open spaces. Their primary function is in-plane drainage. Because of their open structure, geonets often require protection against clogging using geotextiles. The hydraulic conductivity (or transmissivity) of geonets is several orders of magnitude higher than that of gravels.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

Geosynthetic type		Description
Geomembranes		They include relatively impermeable sheets of polymeric formulations used as a barrier to liquids and/or vapors. Data are now available regarding their expected lifetime, which indicate that long-term durability of geomembranes is less of a concern than initially anticipated (significantly longer than concrete, for example). Indeed, even exposed geomembranes are now being used in the design of waste containment cover systems.
Geocomposites		They represent a subset of geosynthetics whereby two or more individual materials are utilized together. They are often laminated or bonded to one another in the manufacturing facility and are shipped to projects as completed units. Geotextile/geonet composites are common. The geotextile serves as both a separator and a filter while geonet serves as a drain.
Geosynthetic Clay Liners (GCLs)		They constitute a composite material used as hydraulic barrier, which involves a thin layer of bentonite sandwiched by other geosynthetics. The geosynthetics are either geotextiles or a geomembrane. With geotextile-encased bentonite, the bentonite is contained by geotextiles on both sides. The geotextiles are bonded using adhesives, by needle-punching, or by stitch-bonding.

The many types of geosynthetics have been used in multiple geotechnical, environmental, and hydraulic applications. Examples of such applications are illustrated in Figure 3, which shows the construction of a geosynthetic-reinforced steep slope to widen an existing road and in Figure 4, which shows the lining of side-slopes of a waste containment facility using geomembranes and GCLs. In some cases, a given geosynthetic may serve multiple functions (e.g. a geocomposite layer that may provide in-plane drainage and also provide protection to an underlying geomembrane). On the other hand, structures such as modern landfills often involve the use of a huge number of geosynthetics. Virtually all types of geosynthetics can be used to perform a significant number of functions in a modern landfill (Figure 5).

Figure 3. View of a geosynthetic-reinforced embankment in a transportation project.

Figure 4. View of a liner system composed of geomembranes and GCLs in a waste containment project.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

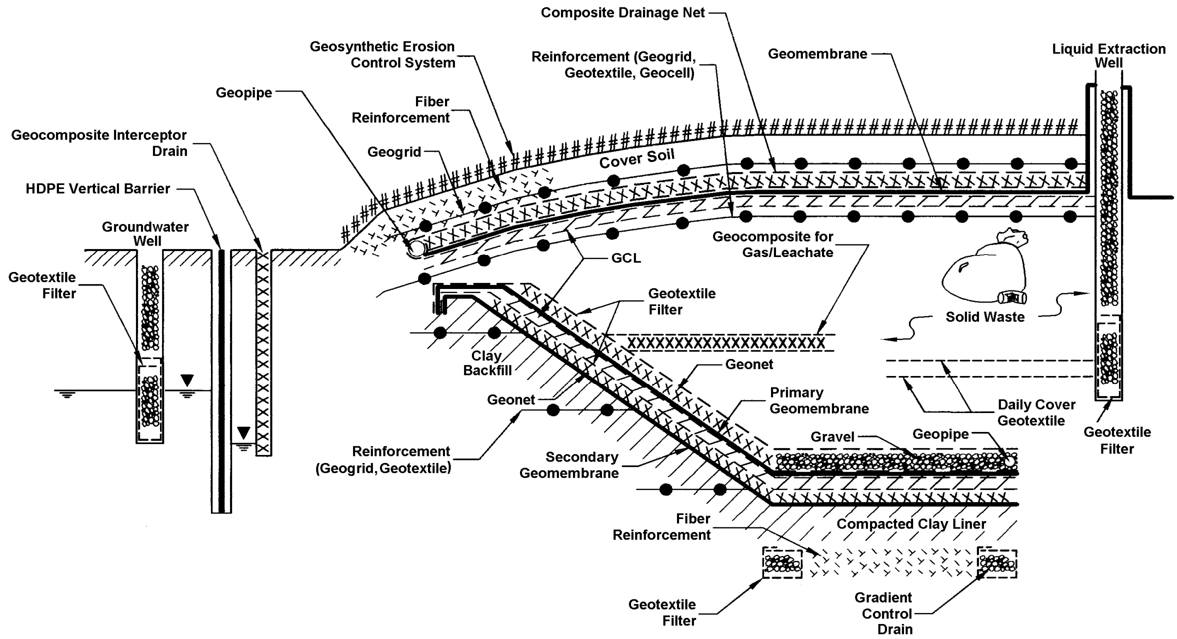


Figure 5. Multiple use of geosynthetics in landfill design (Zornberg and Christopher 2007)

Common Knowledge Base of Geotechnical and Geosynthetics Engineering

Unlike soils and rocks, geosynthetics are geomaterials manufactured in controlled environments that generally include strict quality control procedures. Yet, the material characterization of geosynthetics as well as the behavior of geosynthetics draws significant similarities with those of soil and rock. There are plenty of examples of such common knowledge base, so this section only provides examples drawn from the experience of the authors.

One such example is the characterization of the soil-geosynthetic interfaces, which can be evaluated within frameworks already developed to characterize the shear behavior of soils. Aspects of the soil-geosynthetic interface behavior such as their drained and undrained response, characterization of the peak and residual interface shear strength, and plastic deformations along these interfaces can be evaluated using the concepts of soil behavior. The specimen configuration used to test the internal shear strength of GCLs is shown in Figure 6 (Zornberg et al. 2005). As shown in the figure, the specimens are placed in a testing device similar to that used for direct shear testing of soils, although the geosynthetic is constrained by bonding the two carrier geotextiles to porous rigid substrates using textured steel gripping surfaces. Shearing is conducted after GCL conditioning by applying a shear load under a constant shear displacement rate. The fact that this geomaterial (i.e. GCL) is manufactured under controlled conditions is reflected by the good repeatability of test results, as shown in Figure 7. The results shown in the figure correspond to the internal shear of GCLs with needle-punched woven and nonwoven carrier geotextiles. However, it should be noted that these results were obtained using specimens collected from a single manufacturing lot and tested with the same conditioning procedures. As with the characterization of internal shear strength of GCLs, many other testing procedures now used for geosynthetics have evolved from concepts originally used for characterization of soils.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

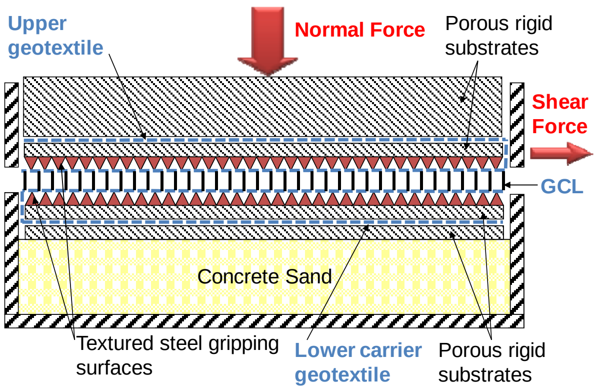


Figure 6. Large-scale direct shear device used for internal shear strength testing of GCLs (Zornberg et al. 2005).

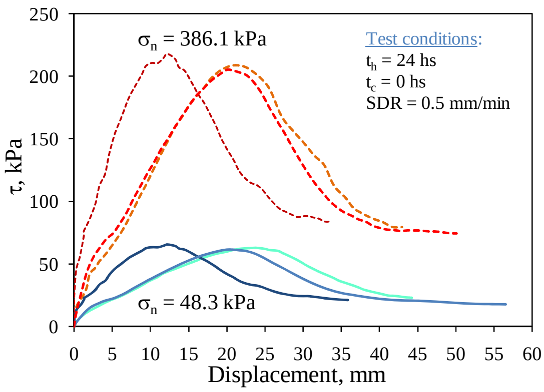


Figure 7. Repeatability of test results on needle-punched GCL specimens using rolls taken from the same lot (Zornberg et al. 2005).

Another good example of common knowledge base between geotechnical and geosynthetics engineering is the framework developed to understand the rate-dependent behavior of clays and geosynthetics. Specifically, Figure 8 shows the results from a drained consolidated triaxial compression (CD TC) test on undisturbed stiff clay that has been subjected to several stages of creep loading and to changes in the applied strain rate (Tatsuoka et al. 2008). In this figure, the prediction of the clay response obtained using an elasto-viscoplastic model is also presented. Similarly, Figure 9 shows a tensile test of a polyester geosynthetic reinforcement that involves monotonic loading at a constant strain rate. However, stages of creep loading were conducted during 30 days at an intermediate stage of the test (Tatsuoka 2008). Also in this case, prediction of the test response was conducted using an elasto-voscoplastic model, and the results of this prediction are shown in the figure. In the case of time-dependent behavior, the theoretical framework originally developed for the understanding of the rate-dependent behavior of soils was useful for the understanding of the creep deformation of geosynthetics as well as of the effect of this behavior on the tensile strength of these materials.

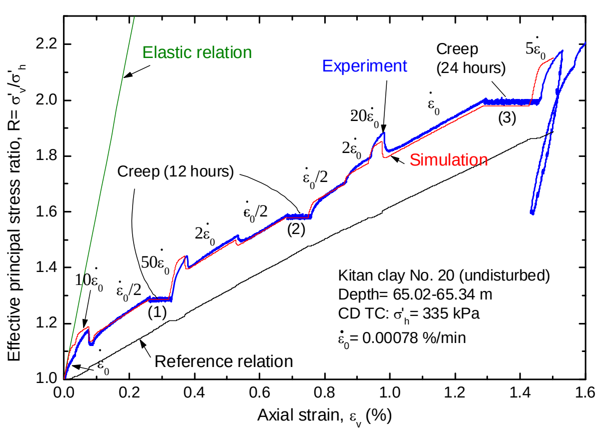


Figure 8. Test results in the pre-peak regime of undisturbed Pleistocene clay in CD TC and its simulation by the three-component model (Tatsuoka et al. 2008)

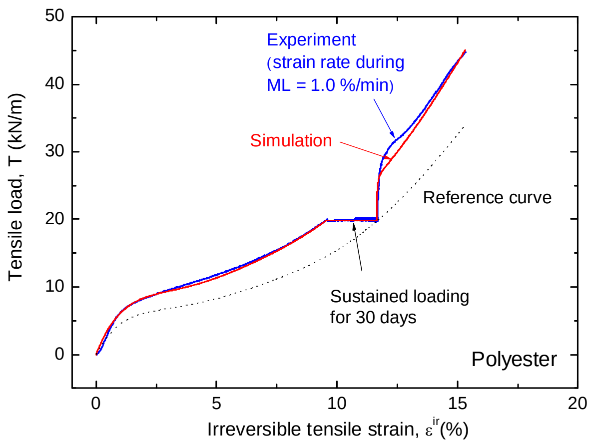


Figure 9. Test results from tensile monotonic loading of a PET geogrid, including a 30 day-long creep stage and its simulation by an elasto-viscoplastic model (Tatsuoka 2008).

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

A Brief History of the IGS

An early international conference on the use of “fabrics” in geotechnics was held in Paris, France, in 1977. However, the concept of an international society, which will later become the IGS, was only formulated in 1980. Subsequently, during a second International Conference on Geotextiles held in Las Vegas, USA in 1982, the formation of the IGS was explicitly discussed. Finally, the “International Geotextile Society,” as it was named at the time, was officially founded on November 10, 1983 with Charles Schaerer as its President. The subsequent presidents were elected by direct vote of each one of the IGS members, including J.P. Giroud (USA), Kerry Rowe (Canada), Colin Jones (UK), Richard Bathurst (Canada), Daniele Cazzuffi (Italy), Fumio Tatsuoka (Japan) and, as of May 2010, Jorge G. Zornberg (USA).

During its 27 years of existence, the IGS has grown remarkably. As previously shown in Table 1, the IGS has 2,263 individual members (compared to 1,869 in 2006) and 141 corporate members (compared to 112 in 2006). The IGS chapters, which are somewhat equivalent to the member societies of the ISSMGE, initiated in Western Europe, North America and Eastern Asia, but later spread out to the rest of the world, including South America, Africa, and Eastern Europe. Growth of the IGS in the form of new chapters is expected to continue in the coming years.

The IGS has held regular International Conferences on Geosynthetics (the ICG) every four years. Subsequent to the aforementioned conferences in Paris and Las Vegas (which indeed took place before the creation of the IGS), the subsequent international conferences were held in Vienna, Austria (1986), The Hague, Netherlands (1990), Singapore (1994), Atlanta, USA (1998), Nice, France (2002), Yokohama, Japan (2006), and finally in Guarujá, Brazil (May 2010). Figure 10 illustrates the location of the various international conferences (and the conference number), as well as the countries in which the IGS has formed local chapters. As illustrated in this figure, the IGS is well represented across the Globe. The next International Geosynthetics conference, the 10ICG, will be held in Berlin, Germany, in 2014. Considering the success of the previous international conferences, we are already looking forward to the many innovations on geosynthetics we expect will be presented in Berlin.

The IGS also organizes regional conferences, which are now well established in the various continents. The last cycle of IGS Regional Conferences included the First Pan-American Geosynthetics Conference (GeoAmericas 2008), held in Cancun, Mexico, the Fourth European Geosynthetics Conference (EuroGeo4) held in 2008 in Edinburgh, UK, the Fourth Asian Geosynthetics Conference (Geosynthetics Asia 2008) held in Shanghai, China, and the First African Geosynthetics Conference (GeoAfrica 2009) held in Cape Town, South Africa. We are also looking forward to the next cycle of IGS Regional Conferences, which are already scheduled to take place in Lima, Peru (GeoAmericas 2012), Valencia, Spain (EuroGeo5 in 2012) and Bangkok, Thailand (Geosynthetics Asia 2012). The series of international and regional conferences of the IGS is complemented by a significant number of national geosynthetics conferences organized by the IGS chapters.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

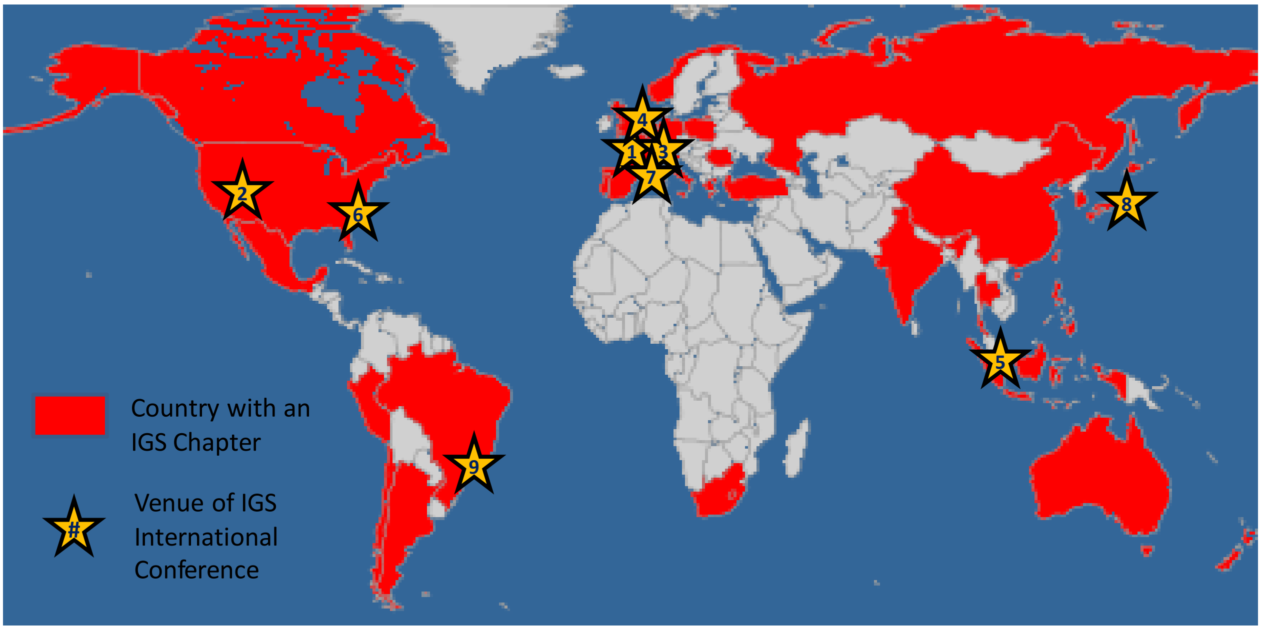


Figure 10. Presence of the IGS in the form of IGS Chapters and location of International Geosynthetic Conferences

Benefits of IGS membership include reduced registration fees when attending any of the IGS international, regional, and national conferences. In addition, the IGS organizes a number of additional programs for its members, including a series of awards program aimed at rewarding technical excellence, service to the IGS and its chapters, and a special program focusing on students. The website of the IGS has been recently revamped, and you are encouraged to visit us at www.geosyntheticssociety.org. Figure 11 shows a view of the home page of the IGS website, which provides access to a significant number of free information, as well as information saved in the 'members only' section. This section includes additional benefits to IGS members (e.g. access to the IGS membership directories, educational presentations, access to our prestigious journals).

The IGS conducts a number of activities aimed at promoting education on geosynthetics. This includes the preparation of training courses or introductory symposia on Geosynthetics Engineering, which have been typically held in conjunction with IGS regional and international conferences. Also, a significant number of two-page IGS educational leaflets on geosynthetic applications have been prepared not only in English but also Portuguese, Spanish, Japanese, Chinese, French, and Italian, among other languages. These leaflets are available for free download at the IGS website.

Dissemination of information on geosynthetics is a core mission of the IGS, and this is well accomplished by the two official technical journals of the IGS: "Geotextiles and Geomembranes" and "Geosynthetics International." We are very proud the extremely high impact factor of our two journals. We should emphasize that access to these two journals is free for IGS members for direct download through the IGS website. This is probably one of the most important direct benefits of the IGS membership. In addition, the newsletter of the IGS, IGSNews, which includes the most updated information about the geosynthetics industry and our Society, has been published quarterly since 1985. IGSNews is also readily available for download at the IGS website.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

www.geosyntheticssociety.org

Figure 11. Portal to the recently revamped IGS website

Cooperation with the ISSMGE

The ISSMGE and IGS have a long history of close cooperation at the domestic, regional and international levels. Many of the IGS chapters have been working closely with the corresponding ISSMGE member societies. In addition, many of the national, regional and international conferences, symposia and workshops organized by ISSMGE or its member societies have been organized under the auspices of the IGS or of the IGS Chapters. Good examples of such collaboration include the series of international conferences on Soil Reinforcement (IS-Kyushu) as well as the support offered by the IGS to the Environmental Geotechnics Congresses organized by ISSMGE. In turn, ISSMGE has offered support to the IGS international conferences. This has included the 8ICG held in Yokohama, Japan (2006) and the recent 9ICG held in Guarujá, Brazil (2010), both of which were held in the support of the ISSMGE. In all these cases, the same reduced registration fee was equally enjoyed by ISSMGE and IGS members. Also, many of the activities conducted by IGS chapters have traditionally received the auspices of the ISSMGE member societies, and vice versa.

It has been very common for IGS members to be strong contributors to the activities of the ISSMGE. This is certainly the case for the authors of this article, who have served as Vice-President for Asia of the ISSMGE and President of the Japanese Geotechnical Society (in the case of IGS Immediate Past-President Tatsuoka); and who currently serve as chair of the International Activities Council of the Geo-Institute of ASCE (in the case of IGS President Zornberg).

On February 21, 2010, the IGS Officers met with ISSMGE President Jean-Louis Briaud and discussed venues of continued collaboration between the ISSMGE and the IGS. Of course, the support to relevant technical events (e.g. technical conferences) will continue. In addition, activities to be jointly conducted by our Technical Committees was identified as new, promising venues to reach our common goals of education, technical excellence, and service to the society at large.

Message to ISSMGE from the President of IGS (continued)

Jorge ZORNBERG and Fumio TATSUOKA

References

Brandl, H. (2010). “Geosynthetics Applications for Mitigation of Natural Disasters.” Giroud Lecture, 9th International Conference on Geosynthetics, Guarujá, Brazil, May, Vol. 1, pp. 379-400.

Giroud, J.P. 2008. The geosynthetics discipline: achievements and challenges. Proceedings of GeoAmericas 2008, First PanAmerican Geosynthetics Conference and Exhibition, Cancún, Mexico: 1-3.

Hudson, J.A. (2010). “Message to ISSMGE from the President of the International Society for Rock Mechanics (ISRM).” ISSMGE Bulletin, Vol.4, no. 1, March, pp. 1-6.

M*ayne, P.W., Coo***p, M., Springma**n, S., Huang, A.B., and Zornberg, J.G. (2009). “Geomaterial Behavior and Testing.” State-of-the-Art Keynote Lecture, Proceedings of the Seventeenth International Conference on Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt, 5-9 October, pp. 2777-2872.

Tatsuoka, F., Di Benedetto, H., Enomoto, T., Kawabe, S. and Kongkitkul, W. (2008). “Various viscosity types of geomaterial in shear and their mathematical expression, Soils and Foundations, *Vol.48, no.1, 41-60.*

Tatsuoka, F. (2008): "Geosynthetics Engineering, combining two engineering disciplines", Special Lecture, Proc. GeoSyntheticsAsia, Shanghai, *Vol. 2, pp.1-35.*

Zornberg, J.G., McCartney, J.S., and Swan, R.H. (2005). “Analysis of a Large Database of GCL Internal Shear Strength Results.” Journal of Geotechnical and Geotechnical Engineering, ASCE, March, Vol. 131, No. 3, pp. 367-380.

Zornberg, J.G., and Christopher, B.R. (2007). “Chapter 37: Geosynthetics.” In: The Handbook of Groundwater Engineering, 2nd Edition, Jacques W. Delleur (Editor-in-Chief), CRC Press, Taylor & Francis Group, Boca Raton, Florida.

New Vice President for Asia

Professor Zuyu Chen, who has been the Vice President of ISSMGE for Asia since 2009, accidentally got injured at the Shenzhen Airport during his trip for Beijing. Considering that his physical recovery will take a long time, Prof. Chen decided to resign his position of vice president. The international president, Prof. Briaud, regretfully decided to accept his decision and, after consulting Asian member societies, nominated Prof. Askar Zhussupbekov from Kazakhstan as the new vice president for Asia. Prof. Zhussupbekov has been one of the appointed board members of ISSMGE since 2009. Moreover, Prof. Charles W.W. Ng from Hong Kong, China, was nominated as an appointed board member after Prof. Zhussupbekov. The board of ISSMGE cordially wishes Prof. Chen to recover from his injury and resume his activities within a short time.

Prof. Askar Zhussupbekov

Prof. Charles W.W. Ng

ISSMGE Technical Committees – 2009 - 2013

In order to activate technical committees, ISSMGE has established a task force that is called Technical Oversight Committee (TOC) and reorganized the structure of Technical committees (TCs). TOC is chaired by Dr. Suzanne Lacasse. ISSMGE wishes to encourage a wide range of activities of TCs in both academism and practices. In particular, TC chairpersons are given with strong power and responsibility for successful management of the committee activities.

Under the new scheme, TCs are classified into three groups and each TC has liaisons from TOC for a closer relationship. Information for new TCs is shown in what follows.

Category	TC #	TC Official Name	Host Country	TC Chair	TC Vice-Chair
Funda-mentals	TC 101	Laboratory Stress Strain Strength Testing of Geomaterials	France	H. di Benedetto	S. Shibuya (Japan)
	TC 102	Ground Property Characterization from In-Situ Tests	USA	P. Mayne	
	TC 103	Numerical Methods in Geomechanics	Hong Kong	K.T. Chau	
	TC 104	Physical Modelling in Geotechnics	Switzerland Hong Kong	S.Springmann (until1 July 2010) C.W.W.Ng	C. Gaudin (Australia)
	TC 105	Geo-Mechanics from Micro to Macro	UK	M. Bolton	
	TC 106	Unsaturated Soils	Spain	E. Alonso	
	TC 107	Laterites and Lateritic Soils	Ghana	K. Ampadu	
Applica-tions	TC 201	Geotechnical Aspects of Dykes and Levees, Shore Protection and Land Reclamation	Netherlands	M.A. Van	
	TC 202	Transportation Geotechnics	Portugal	A. Gomes Correia	
	TC 203	Earthquake Geotechnical Engineering and Associated Problems	Greece	K. Pitilakis	
	TC 204	Underground Construction in Soft Ground	France	R. Kastner	
	TC 205	Limit State design in Geotechnical Engineering	UK	B. Simpson	
	TC 206	Interactive Geotechnical design	Canada	K. Been	

ISSMGE Technical Committees – 2009 – 2013 (continued)

Category	TC #	TC Official Name	Host Country	TC Chair	TC Vice-Chair
Applica-tions	TC 207	Soil-Structure Interaction and Retaining Walls	Russia	V. Ulitsky	
	TC 208	Stability of Natural Slopes	Canada	J. Fannin	
	TC 209	Offshore Geotechnics	USA	P. Jeanjean	
	TC 210	Dams and Embankments	China	Dr. Xu Zeping	
	TC 211	Ground Improvement	France	S. Varaksin	
	TC 212	Deep Foundations	Germany	R. Katzenbach	
	TC 213	Geotechnics of Soil Erosion	Germany	M Heibaum	
	TC 214	Foundation Engineering for Difficult Soft Soil Conditions	Mexico	J.L. Rangel	
	TC 215	Environmental Geotechnics	Italy	M. Manassero	
	TC 216	Frost Geotechnics	Norway	A. Instanaes	
Impact on society	TC301	Preservation of Historic Sites	Italy	C. Viggiani	Y. Iwasaki (Japan)
	TC302	Forensic Geotechnical Engineering	India	V.V.S. Rao	
	TC303	Coastal and River Disaster Mitigation and Rehabilitation	Japan	S. Iai	
	TC304	Engineering Practice of Risk Assessment and Management	Singapore	K.K. Phoon	
	TC305	Geotechnical Infra-structure for Megacities and New Capitals	Brazil	A. Negro	
	TC306	Geo-engineering Education (include aspects of software in use)	Australia	M. Jaksa	M. Bouassida (Tunisia)
	TC-307	Dealing with sea level changes and subsidence	Southeast Asia	Thiam-Soon Tan (Singapore)	

DR. OSCAR A. VARDÉ AS THE PRESIDENT OF NATIONAL ACADEMY OF ENGINEERING OF ARGENTINA

On April12, 2010, Oscar A. Vardé was elected President of the National Academy of Engineering (N.A.E.), Argentina. This is the highest recognition an engineer can earn after 50 years of his ceaseless professional activity.

The Geotechnical Community members are proud that someone who has spent his professional activity in the field of our speciality has achieved this distinction. Beside teaching, he has also served as the President of the Argentinian Society of Soil Mechanics (4 periods), Vice- President of the ISSMGE for South America (1985-1989) and the ISRM (1991-1995). He participated in many ISSMGE TC´s and in other Technical and Academic Societies like ITA (International Tunnel Association).

For his outstanding accomplishments, he was honored with the Arthur Casagrande Lecture Award in 1991, Perez Guerra Prize in 1993, and the Konex Prize in 2003. He was compiled on extensive record of publications in top journals, and he is the author of 105 papers published in different events on geotechnical engineering, geological engineering, tunnels, and dams. He was a co-editor of the Special Volume on “Pile Foundation and Negative Skin Friction.”

Beyond being curious and restless, his commanding personality, his passion and dedication, together with innate leadership skills, he has proven to be an asset to our society as a whole.

His kindness has earned him the friendship and respect of many colleagues around the world. This is evidenced by many greetings that were received by his recent appointments.

He well deserves this position and we are grateful for his friendship. We wish him the best in years to come, and we are proud to be among his disciples.

Prof. Jorge Bonifazzi (ISSMGE past VP for South America) and
Prof. Roberto Terzariol (ISSMGE VP for South America)

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake

Susumu Yasuda, Tokyo Denki University, Japan
Ramon Verdugo, Universidad de Chile, Chile
Kazuo Konagai, University of Tokyo, Japan
Takahiro Sugano, Port and Airport Research Institute, Japan
Felipe Villalobos, Universidad Catolica de la Santisima Concepción, Chile
Mitsu Okamura, Ehime University, Japan
Tetsuo Tobita, Kyoto University, Japan
Andres Torres, Universidad de Chile, Chile
Ikuo Towhata, University of Tokyo, Japan

1. INTRODUCTION

The country of Chile is located along the subduction of Nazca tectonic plate, that is moving at the rate of 7 cm/year , against South America. Consequently, many gigantic earthquakes have affected this country in the past. For example, the event in 1960 registered 9.5 in magnitude and caused significant damage in its south part including Valdivia. Another event in 1985 was of magnitude of 7.8 and affected the central part of the country where Valparaíso and San Antonio are situated (Troncoso, 1989).

On February 27th, 2010, the Maule earthquake of 8.8 in moment magnitude occurred in a region between those two former earthquakes. The epicenter was located near Cobquecura. The length of the seismogenic fault was estimated to be around 450 to 500 km along the Pacific Coast. This means that Santiago, the capital, and Valparaíso are located near the north boundary of the fault, while Concepcíon near the south end. Cities in the affected area are situated either in the central valley (Quaternary geology in Fig. 1) or along the Pacific Coast. Buildings, houses, bridges, road embankments, tailing dams and other structures were damaged by the earthquake.

Figure 1 illustrates the geological condition in the affected area. The coastal mountain range along the Pacific Ocean are covered by Tertiary deposits, while Andes Mountains, which is parallel to the coast, is made of Tertiary and older rocks. The central valley between these two mountain ranges has a deposit of Quaternary soil. Quaternary soil deposit is also found in several lowlands along the Pacific Coast. However, those lowlands are very small except the one around Concepción.

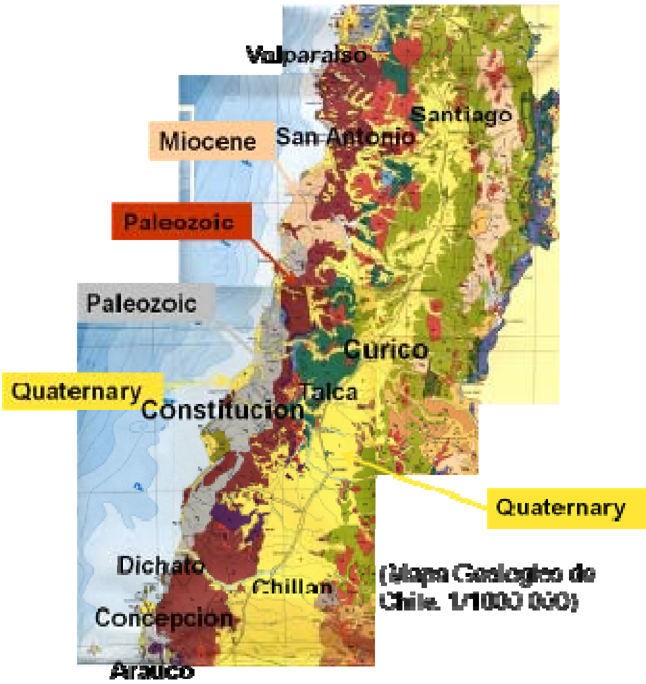


Fig. 1 Geological map in the affected area by the earthquake

A cold sea current in the Pacific Ocean makes the climate in the affected region relatively dry. In particular, the northern part receives less precipitation than the southern part. Typically, the average annual rainfall in Santiago is only 350 mm/year.

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

About one month after the earthquake (late March to early April), four engineering societies in Japan, which were namely Japan Association of Earthquake Engineering, Japanese Geotechnical Society, Japan Society of Civil Engineering, and Architectural Institute of Japan, jointly dispatched a reconnaissance team to investigate the damages in collaboration with specialists in Chile. This article was written as a partial product of this joint activity and focuses on geotechnical issues.

2. LIQUEFACTION-INDUCED DAMAGE

Soil liquefaction occurred at a variety of places because the magnitude of the earthquake was large. The greatest epicentral distance of liquefaction site is 300 km at Veta del Agua tailing dam in the northern part. This farthest distance is plotted against the magnitude in Fig. 2 to show good compatibility with experiences during past earthquakes.

The actual number of liquefied sites was quite small in spite of the large earthquake magnitude. This is probably because the low level of annual precipitation does not make many water-saturated sandy subsoil. The reconnaissance survey detected liquefaction at fill or replaced backfill in and around Concepción only, except the tailing dam at Las Palmas. In Concepción area, subsoil liquefaction caused settlement of buildings and houses as well as uplift of underground tanks at sites shown in Fig. 3. Photo 1 demonstrates a 0.77-degree tilting of an 8-storied apartment building at Los Presidentes in Hualpén. Sand boils were found around the building to verify the occurrence of liquefaction. As the original topography here was swamp, subsurface soil was excavated down to the depth of 4 m and then sand was placed at the time of construction (see Fig. 4). Ground water table was as shallow as 1.0 m below the surface, and it made the effects of subsoil liquefaction more influential. Similar settlement occurred to a 5-storied hospital building in Curanilahue at about 30 km south of Concepción.

Many houses settled in three housing lots in Concepción as shown in Photo 2. The maximum settlement was about 17 cm. Because the original topography here was swamp as well, the backfilled sand in the swamp liquefied and caused house to subside. Furthermore, underground tanks uplifted in cities of Concepción, Chillan, and Arauco. Photo 3 shows uplifted sewage tanks. It seems that backfilled sand liquefied and caused this uplift.

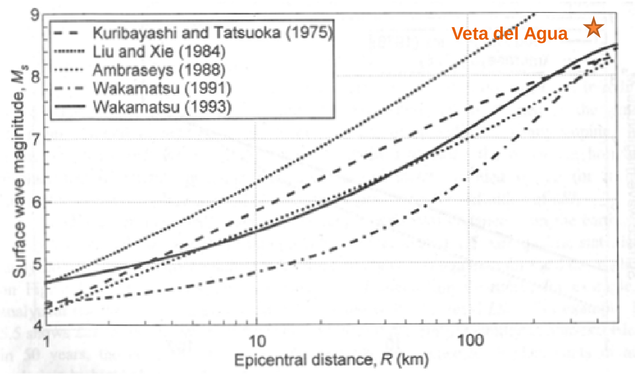


Fig. 2 Relationship between seismic magnitude and epicentral distance at farthest liquefaction sites

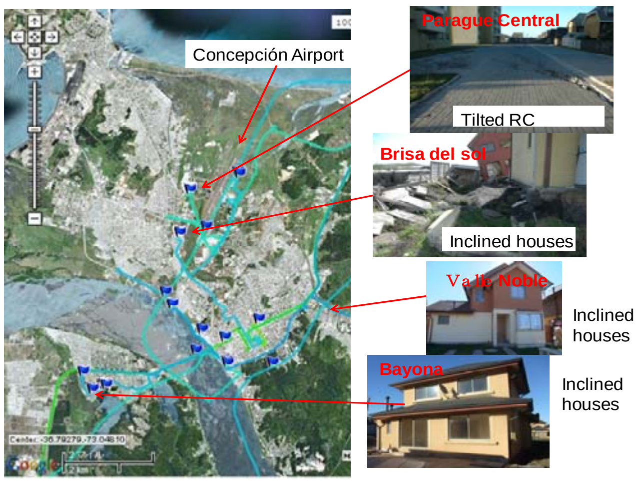


Fig. 3 Sites of liquefaction damage in Concepción

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Photo 1 Tilting of apartment building at Los Presidentes

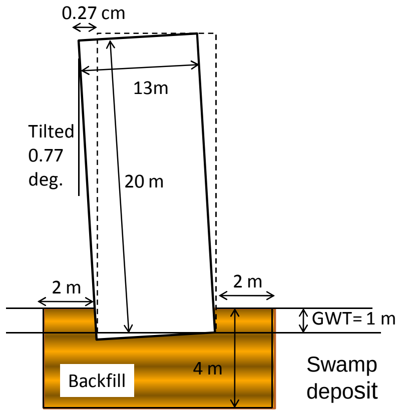


Fig. 4 Schematic cross section of tilted building

Photo 2 17-cm Subsidence of houses in Bayona, San Pedro de la Paz

(Densification by dynamic compaction)

Photo 4 Apartment buildings without damage in Concepción

Photo 3 Uplift of buried sewage tank in San Pedro de la Valle

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

On the contrary apartment buildings in Photo 4 survived the quake although liquefaction occurred in its neighborhood. This is because the foundation soil of these buildings had been densified by dynamic compaction method and liquefaction resistance had been increased. This is an important evidence to validate the effects of this soil-improvement technology.

3. DAMAGE IN DAM

3.1 Liquefaction in Tailing Dam

Tailing is a waste material that is produced by mining industries. Valuable minerals are removed from powder of ores and the remaining stone powders are dumped into a water pond (Photo 5). Since the powders are as fine as silt, they sediment in water very slowly and form a loose and liquefiable deposit. Further problem is that this fine grain size reduces the permeability (hydraulic conductivity) of this material and, in case of liquefaction and high excess pore water pressure, the dissipation of high pore pressure is substantially delayed. Therefore, the adverse condition of high pore water pressure and low effective stress lasts for a long time. Moreover, this fine grain has no cohesion, thus increasing the liquefaction probability further. The entire tailing deposit is supported by an embankment made of coarse components of tailing as well. In case of up-stream construction, this embankment is nothing more than a surface coverage, and if the underlying tailings get liquefied, the coverage cannot maintain stability anymore. Hence, a tailing flow occurs.

Fig. 5 Location of liquefaction of mine tailing.

Liquefaction of mine tailings occurred at Las Palmas near Curico, Veta del Agua, and La Florida (Fig. 5). Among them, the authors were able to visit the significant damage at Las Palmas. This tailing dam was used for wastes from a gold mine between 1981 and 1997. An abandoned tailing dam collapsed as shown in Photo 6. Photo 7 illustrates details of the collapsed dam. Accordingly, water and liquefied tailings erupted from many cracks at the surface (see Photo 8). The liquefied mine tailings flowed down about 400 m and hit a farmer’s house (Photo 6). Consequently, four people were buried to death under the tailing mass.

Photo 5 Reservoir of tailing dam at Veta del Agua in 1993.

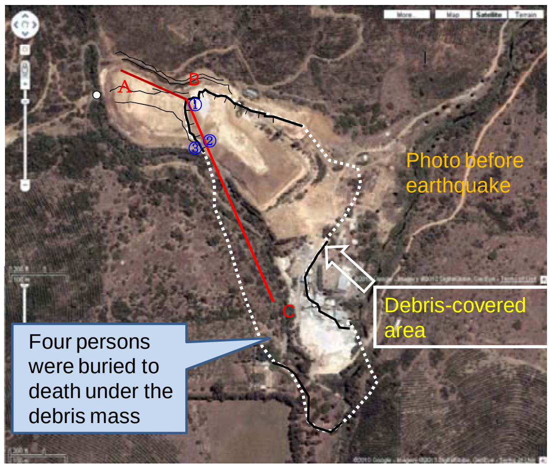


Photo 6 Extent of damage at Las Palmas tailing dam.

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Photo 7 Tailing deposit after liquefaction

Photo 8 Eruption of liquefied tailings at Las Palmas

3.2 Damage of Water Dam

Coihueco Dam is situated at around 130 km to the east of Concepción. It is an earth dam and measures 975 m in length, 31 m in the maximum height, 19 degrees in the upstream slope, and 21 degrees in the downstream slope. The reservoir area is 2.26 million m2. Photo 9 indicates two longitudinal cracks along the crest of the dam; one in the centre and the other at the downstream (right side) shoulder. The width of the crest is 5.2 m. The depth of the crack reached at maximum 1.9 m. Photo 10 demonstrates the downward slip movement of the slope on the reservoir side. This movement resulted in at maximum 3.3 m subsidence near the shoulder.

Photo 9 Longitudinal cracks at the crest of Coihueco Dam

Photo 10 Downward slope movement in upstream face

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

4. FAILURES OF SLOPES AND EMBANKMENTS

It was fortunate that the size of failures in natural slopes was small. This is because the Andes Mountains where there are many unstable slopes were far from the earthquake-affected area. The Coastal Mountains conversely do not have many steep slopes except cliffs at the coast. Therefore, the present report on natural terrain addresses only the coastal region. There are many steep cliffs along the coast from Arauco to Lebu through Cape Lavapie and they are subject to shallow failures. Photo 11 and Photo 12 indicate surface failures near Arauco and Cape Lavapie, respectively. Height of the cliff shown in Photo 11 is about 100 m.

Photo 11 Slope failure at Las Peñas near Arauco

Photo 12 Slope failure at Lavapié

Photo 13 Minor slope failure at San Antonio

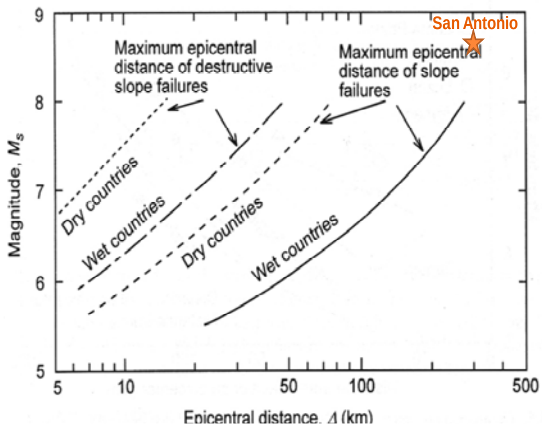


Fig. 6 Relationship between magnitude and epicentral distance to farthest landslide sites (TC4 of ISSMGE, 1999)

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Photo 14 Peaty ground in and around Tubul

Photo 15 Collapsed bridge in Tubul

Photo 16 Collapsed approach embankment of Raqui II bridge

Photo 17 Big slide of road embankment in Lota Norte

Although being very minor, a slope failure with the longest epicentral distance occurred in San Antonio (Photo 13). Its epicentral distance of about 300 km is plotted against the seismic magnitude in Fig. 6 to support the relationship between magnitude and maximum distance of slope failure that was proposed in the Seismic Zoning Manual by TC4 (TC4, 1999). The plotted point is between two curves for dry countries and wet counties. Because the slope failure in Photo 13 is very minor, it may be reasonable to shift the point to the left.. Anyway, although the magnitude of the earthquake was huge, the size of slope-failure area was small as compared with that in wet counties.

5. DAMAGE OF BRIDGE AND EMBANKMENT RESTING ON SOFT GROUND

The subsoil condition between Arouco and Tubul consists of very soft and peaty soil (Photo 14). Consequently, three bridge girders fell down (Photo 15) and approach embankments collapsed as shown in Photos 16 and 17. The mechanism of the collapse of girders needs further investigation, but the effects of soil condition deserve careful study.

In Chile, road embankment is generally constructed only at approaches to bridges. Thus, damaged embankment was found at a limited number of places. Photo 17 shows the biggest failure of road embankment that occurred in Lota. This embankment was constructed upon a swampy soil with a height of 16 m. Its construction material was clean sand. It seems therefore that pore water pressure increased at the bottom of the embankment during shaking, leading to loss of shear strength and shear failure.

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

6. DAMAGE IN HARBOR STRUCTURES

Coronel Harbor is located at about 450km to the south of Valparaiso. A distortion occurred in a fishermen's pier there (Photo 18). The damaged structure connects the land and the main part of the harbor and is supported by a pile foundation. Because substantial translation and tension cracks were found on the land side of the pier (Photo 19), it is inferred that the structure was subjected to compression from the land side, and, because of the very rigid foundation of the main part of the offshore pier, the connecting part developed significant compressional deformation and buckling. This soil-structure interaction deserves further study. Note that two larger commercial piers next to this place received only minor effect from the earthquake and were able to start operation one day after the earthquake.

Photo 18 Damaged pier in Coronel

Photo 19 Lateral flow of soil adjacent to the pier

7. REMARKS ON TSUNAMI EFFECTS

Tsunami disaster was reported widely after the earthquake. The height of the wave was 5.6 to 28.3 m in Constitución, 5.3 to 7.3 m in Dichato, 2.8 to 6.4 m in Talcahuno, and 5.2 m in San Antonio. High waves claimed significant human loss and destroyed many structures. One of the reasons for different tsunami heights at different places is the local topography. This section addresses the Coliumo Bay and Dichato area for example (Fig. 7). Photo 20 shows the total devastation near Cape Blanca. This damage occurred on the southeastern side of the cape facing the Coliumo Bay. This is in a clear contrast with the Pacific Ocean side of the cape where no significant damage occurred. It deserves attention that a sea wall in this damaged area functioned satisfactorily (Photo 21).

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Fig. 7 Map of Dichato and Coliumo Bay area


Photo 20 Tsunami damage near Cape Blanca


Photo 21 Successful performance of sea wall in Cape Blanca area.


Photo 22 Tsunami erosion in coastal area of Dichato

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Photo 23 Tsunami-induced erosion in Dichato

Photo 24 Exposure of buried lifeline after tsunami erosion

Photo 25 House that floated when tsunami came

Photo 26 House that did not float when tsunami came

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

Photo 27 Tsunami protection by embankment (drawn on Google Map)

Photo 22 indicates a long-distance view of Dichato. Although it is not visible in this photograph, the coast of this town was eroded by the tsunami as shown in Photo 23. It is noteworthy that even an underground facility was affected by tsunami-induced erosion of soil; see Photo 24. Photos 25 and 26 demonstrate effects of foundation on damage extent of houses. The house in Photo 25 floated when the water level rose and was transported over a long distance. In contrast, the house in Photo 26 was tightly connected to the foundation and was able to stay in the same place. The intact shape of this house may suggest that the impact force of the tsunami was not very strong. Finally, Photo 27 presents an interesting case where a 6-m-high road embankment protected a freight container yard from the tsunami attack.

Case History

Geotechnical damage caused by the 2010 Maule, Chile earthquake (continued)

8. REMARK ON BUILDING DAMAGE

Several buildings collapsed in Santiago, Curico and Concepción. Some of them had weak pillars as shown in Photo 28. When substantial inertia force occurred in the massive superstructure, the ground floor, that consisted only of pillars without a reinforced wall, was easily destroyed. This type of structural failure has been experienced in many past earthquakes.

Photo 28 Collapsed building at Maipú in Santiago

9. CONCLUDING REMARKS

The authors conducted a damage reconnaissance study in Chile after the gigantic Maule earthquake in 2010. Although the magnitude of the earthquake was as large as 8.8, damage to structures was limited except tsunami-induced ones. In particular it was fortunate that liquefaction and landslide occurred at few sites only. On the other hand the importance of soil-structure interaction in damage generation was found in harbour structures and underground lifelines. This issue needs further consideration.

10. ACKNOWLEDGEMENTS

The present study was conducted in collaboration of four Japanese Societies with specialists in Chile. The authors express their sincere thanks to those who assisted this activity. In particular, the kind supports by Prof. Y. Kitagawa of Keioh University, who was the head of the entire investigation team, Prof. S. Midorikawa of Tokyo Institute of Technology, who was the general secretary of the team, and Prof. J.H. Troncoso of Pontificia Universidad Catolica de Chile are deeply appreciated. The authors also express their sympathy to earthquake victims and affected people. It is emphasized here that the engineering community should understand the real damage mechanisms during earthquakes and develop necessary provisions for future damage mitigations.

11. REFERENCES

Duke, C. M. and Leeds, D. J., "Response of soils, foundations, and earth structures to the Chilean earthquake of 1960," Bull. of Seismological Society of America, Vol.53, No.2, pp.309-357, 1963.

Troncoso, J.H., "The Chilean earthquake of March 3, 1985: Effects on Soil Structures," Proc. of Discussion Session on Influence of Local Conditions on Seismic Response, 12th ICSMFE, pp.1-10, 1989.

TC4, ISSMGE., "Manual for Zonation on Seismic Geotechnical Hazards (Revised Version)," publ. Japanese Geotechnical Society, 1999.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay

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Peter Boyle
Port of Brisbane Corporation, Fisherman Islands, Brisbane, Australia
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Menard Bachy, Sydney, Australia

The Port of Brisbane (PoB), located at the mouth of the Brisbane River at Fisherman Islands, is undertaking a reclamation expansion of a 235ha sub-tidal area using maintenance dredged materials from the adjacent river. The reclaimed site is underlain by soft dredged materials up to 9m thickness as well as soft to firm Holocene clays in the natural sea bed as deep as 30m. Extensive ground improvement is required for such deposits prior to releasing the land for development. A well planned set of ground improvement trials involving international operators were conducted to optimise and select suitable techniques prior to rolling out. The project won the Innovation Award in 2009 from the Institution of Engineers Queensland Division in Australia. This paper describes the project, characteristics of the PoB clay, the reasons for deciding on trials and a discussion of some results from the trials.

Port of Brisbane

Port of Brisbane (PoB) is the main container port of the State of Queensland on the east coast of Australia (Figure 1). It is located in the lower reaches of the Brisbane River on reclaimed land at the mouth of the river. The land reclamation had been in progress since the early 1980's and the current Port footprint, now called Fisherman Islands, is almost entirely constructed in the adjacent Moreton Bay. Figure 2 shows the history of reclamation in Fisherman Islands and the gradual reclamation and development towards the northeast since 1958.

Figure 1: Site location (Courtesy of Port of Brisbane Corporation)

-

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Figure 2: Fisherman Islands and Port of Brisbane (Courtesy of Port of Brisbane Corporation)

Being the third largest container port in the country, Port of Brisbane has been steadily developing to cater for increasing trade growth. In recent years, there has been a rapid growth in the demand for port land as a result of increased trade growth in the South East Queensland region, which is expected to continue beyond the next 25 years. In 1999, the Port of Brisbane Corporation embarked on the expansion of a 235ha sub-tidal area immediately adjacent and northeast of the existing land. As a first step, a 4.6km long Future Port Expansion (FPE) seawall (Figure 3) was constructed around the perimeter of the site in Moreton Bay so that progressive reclamation works can be carried out within the boundaries of the seawall (Ameratunga et al, 2005).

The area contained by the FPE seawall required that the Port should achieve two objectives:

- To provide land for development to cater for future needs
- To act as a receptacle to dispose of the materials generated from the river maintenance dredging activities in the Brisbane River channel

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Figure 3: Future Port Expansion (FPE) Seawall - 2004

Although the above benefits are significant to the Port, the site posed significant challenges to make the reclaimed land ready for development. The most challenging is the weak and compressible soil formation across the site. The latter can be summarised as Holocene deposits overlying Pleistocene deposits. The Holocene deposits include weak and compressible clays, and occur as paleo-channels across the site. The depth of the Holocene clays (or PoB clays) varies from a few metres to as deep as 30m. The conditions are exacerbated after reclamation, because the reclamation is undertaken using the maintenance dredged materials from the adjacent Brisbane River. These materials are classified as clay or silt and commonly referred to as “dredged mud”, significantly weaker than the underlying PoB clays. The thickness of these layers also varies, up to about 9m across the reclamation areas, leading to overall clay thicknesses of more than 30m. As both in-situ clays and dredged materials are highly compressible, settlement due to filling alone could be as high as 2m even before any service loads are imposed. Improving such deep deposits provided a challenging exercise in ground improvement considering the performance criteria required by the Port of Brisbane Corporation in its short and long term utility.

Therefore it is imperative that the land is improved prior to any development unless expensive solutions such as pile foundations are adopted for all infrastructure, and even then, serviceability could still be an issue. It was estimated that it would take in excess of 50 years to treat this land using conventional reclamation and surcharging methods, due to the existing soft clay depths compounded by the thickness of the overlain dredged mud.

Initial desk studies by Coffey made an assessment of relative merits of the many ground improvement methods that could be used under the site conditions. They included surcharging with wick drains or sand drains, vacuum consolidation, stone columns, deep soil mixing, and controlled modular columns. Preliminary cost estimates were carried out based on rates provided by the industry and/or similar construction works in the State of Queensland or any other State. The results indicated that wick drains would be the least expensive solution for the wider area with more specific solutions to be adopted for the boundaries, i.e., edges abutting the Moreton Bay Marine Park.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Wick drains, to our knowledge, had never been used to improve such deep deposits of clays nor dredged muds in Australia although case histories from overseas are available in the public domain. Initial enquiries from the local market indicated that none of the machines had mast lengths sufficiently long to penetrate the deep clays found at the Port. Furthermore, in South East Queensland, there have been several documented cases of underperformance of wick drains on projects since the late 1980's. There can be several reasons for such underperformance of which the subsoil characteristics are most likely to be one of them. In addition, there were many unknowns, especially relating to the installation technique and the potential smearing of the wick drains. The Port of Brisbane Corporation (PBC) was concerned because a small trial within Fisherman Islands a few years back also provided similar poor results. PBC accordingly required some certainty of performance, including timelines, with respect to the assessment of the effectiveness of the various techniques and to optimise designs before embarking on a full scale treatment of the site, to ascertain the most applicable methods of consolidating the soft clay soils. However, neither PBC nor Coffey wanted to suppress innovation and therefore formulated the trials to allow the ground improvement specialists/contractors to propose their own solutions.

Tenders were called internationally for ground improvement techniques that could be effectively applied over a selected area of the Port as a trial. Once the tenders were closed it was clear that wick drains were nominated as the preferred solution for the main trial areas with vacuum consolidation being nominated by one of the contractors for the edge areas. The trials were expanded so that both types could be trialled at the same time. The trials were completed in December 2008 and provided valuable learnings on wick drain and vacuum treatment in reclamation works.

Geological Setting

The geological setting is described in many of the geotechnical reports related to the development of the Port. A brief description is given below (Ameratunga et al, 2010).

The geomorphology of northern Moreton Bay and the southern Sunshine Coast area has undergone major changes in the last twelve thousand years. This period marks the end of the previous Ice Age at a time when the sea level was around 150m lower than it is today and the coast line was approximately 25km to the east of its current location.

Since then, the bay has been emptied approximately four times and partially in-filled approximately five times in response to world-wide changes in sea level caused by minor Ice Ages at various times. These fluctuations in sea level resulted in a complex series of sediment layers and erosion surfaces; each incursion laid down sediments, which were then partially or totally eroded as the sea level subsequently fell. The remaining sediments were then covered over by subsequent incursions and the cycle continued. At the present day, the sea level is unusually high when compared to the typical sea levels over the previous one hundred and twenty thousand years. In late Pleistocene times, the sea level rose progressively from around 150m below its current level (with a shoreline around 25km east of Moreton Island) approximately 19,000 years ago, to slightly above its current level around 6000 years ago, before settling at its current level. The buried land surface between the older Pleistocene sediments and the more recent Holocene sediments is of particular interest in formulating a geological model for the study area.

The massive barrier islands of North and South Stradbroke, Moreton and Bribie Islands dominate the Moreton Bay area. In the protected landward area of Moreton Bay, there is a general zoning of recent sediments roughly corresponding to the local sedimentary conditions.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Near the mouth of the Brisbane River, the dominant sediments are marine sand, silt and clay deposited from the waters of the Brisbane River as they enter the still water of the bay. Around the coastal areas of the northern suburbs, Redcliffe and Deception Bay, marine muds derived from local terrestrial erosion dominate the sediments. In the northern and southern parts of the bay, sandy sediments are present, which have been washed into the bay by tidal currents.

In the study area, four distinct geological units have been recognised and they are listed from top down in Table 1.

Table 1 Geological Units

Unit	Description
Recent	Marine and dune sands with layers of silt and clay. This material may include fill including dredged fill. It is generally silty clay, although a variation in material characteristics across the paddocks is expected because of the single point discharge system with coarser materials depositing closer to the discharge point.
Holocene	Normally consolidated marine clay, silt and sand. A more detailed description is given later in this paper.
Pleistocene	These are older sediments that lie below the PoB clays or the pre-Holocene land surface and consist of overconsolidated, stiff to very stiff silty and sandy clay with layers of clayey sand. The compressibility of these materials is relatively low compared to the soft/firm clays of the Holocene deposits.
Tertiary	The weathered basalt bedrock of the Petrie Formation underlies the site and is described as grey-green clay (extremely weathered basalt) grading downwards into dark grey to black, moderately to slightly weathered basalt.

<u>Port of Brisbane (PoB) Clay</u>

The depth of Holocene sediments or PoB clays has a significant impact on the development because the clayey materials within this profile are compressible leading to high settlements. The basal contours of the PoB area (Figure 4) show how the paleochannels cut across the site. The base of the layer appears to vary from about RL-5m to deeper than RL -35m.

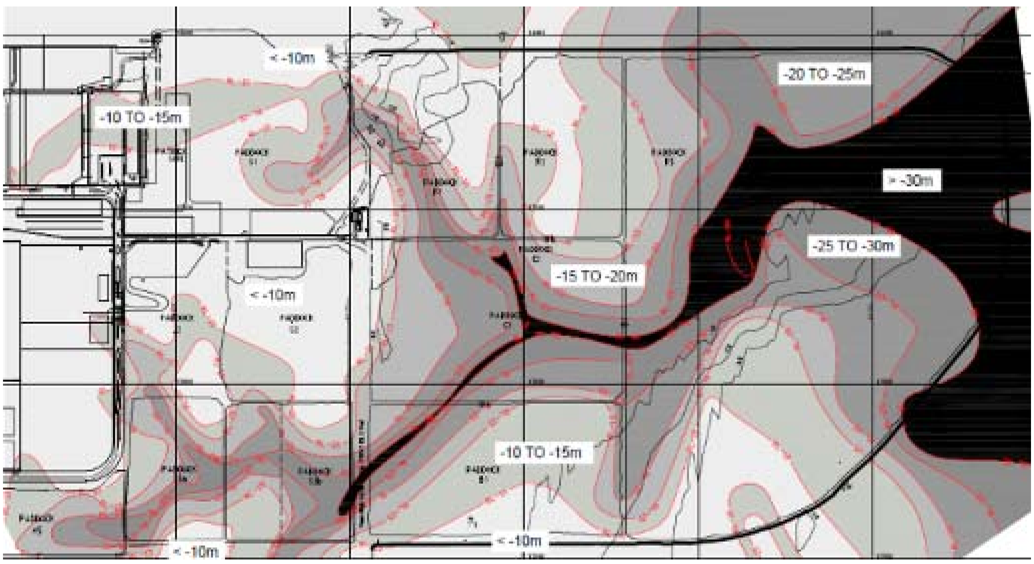


Figure 4: Base contours of PoB clay

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Generally, the Holocene sediments are divided into an upper layer and a lower layer of normally consolidated, low strength silty clay with shell bands ("marine clay"), separated by a discontinuous layer of sand. The upper layer generally consists of sand layers interspersed with layers of soft clays and silts. Sand layers or lenses are relatively few or absent within the lower layer. A typical profile is shown in Figure 5 as a piezocone plot.

The upper layer does not pose significant constraints although it is compressible and leads to settlement. The rate of settlement of this layer is generally rapid because of interspersed sand layers accelerating the dissipation of pore pressures. In the natural soil profile at the site, the lower layer generally controls the rate of settlement because of its greater thickness and the absence of sand layers to accelerate pore water pressure dissipation. Therefore, apart from the reclamation fill, this layer poses the most significant constraint on the development of the land for future use. More attention was therefore directed during the investigation and design phases to understand the behaviour of this lower layer. The bulk of the results presented in this paper are related to this layer where most testing was carried out.

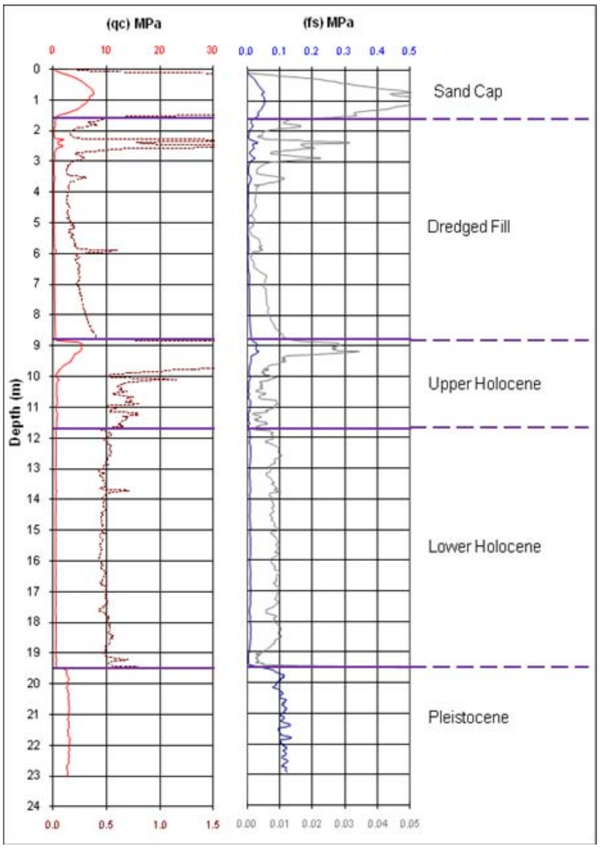


Figure 5: Typical piezocone profile

Index Properties of PoB Clays

PoB clays are found to be highly plastic as can be seen from the Atterberg limit test results plotted on the Casagrande plasticity chart (Figure 6). This figure shows a wide variation of the liquid limit, generally ranging from about 40% to 100%, and the plasticity index generally ranging from 20 to 70. Most importantly, the results clearly show the materials to be clays rather than silts. These laboratory results confirm visual observations on site when clay samples are taken from the PoB clay deposit.

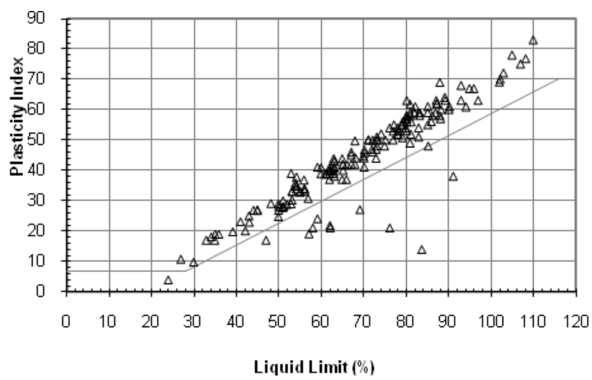


Figure 6: Atterberg limits in Casagrande plasticity chart

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Index tests such as Atterberg limits and the moisture content provide valuable information on the characteristics of a clay. As Balasubramaniam (2010) emphasised, these tests are most valuable when quality undisturbed soft clay samples cannot be retrieved. The results of moisture content, liquid limit, plasticity index and liquidity index are plotted against elevation in Figure 7. The average moisture content appears to be about 60% with the liquid limit slightly above that, indicating possible slight over-consolidation of the soils. The liquidity index is another important parameter which has been plotted on Figure 7 as it provides guidance on how sensitive particular clay is, with higher values greater than 1, indicating higher sensitivity. The liquidity index for the PoB clays was found to be on average less than 1 which indicates the clays may be only moderately sensitive to disturbance. At shallow depth, a more sensitive layer having a liquidity index slightly over 1 was observed but this layer is generally found to be thin.

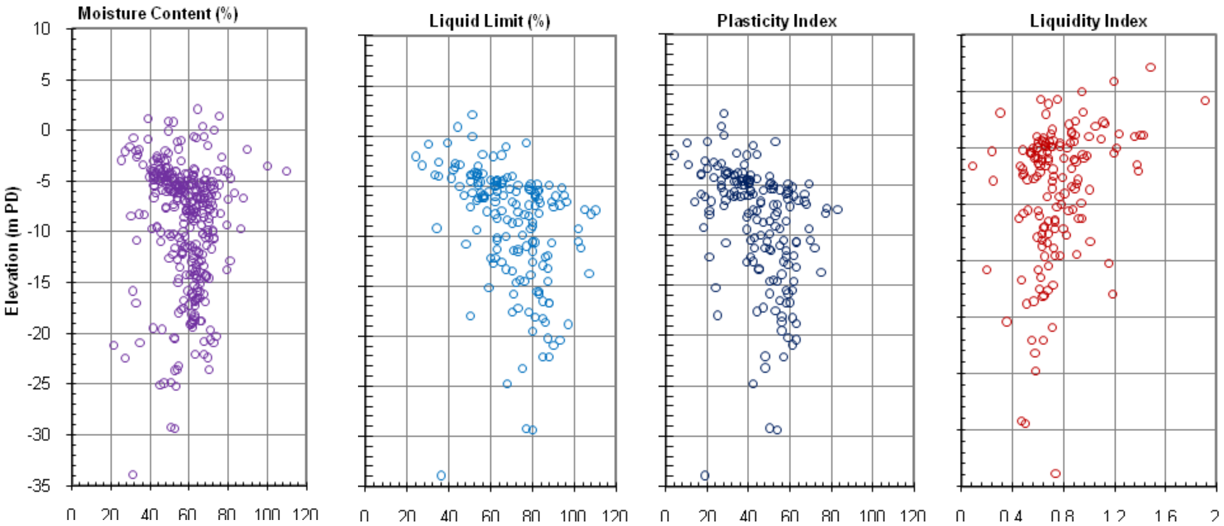


Figure 7 Results of laboratory index tests on PoB clay

Undrained Shear Strength of PoB Clay

It is quite common in Australia to rely on insitu tests to assess the undrained shear strength of a clay deposit rather than laboratory tests such as unconsolidated/consolidated undrained triaxial tests or unconfined compression tests. For softer soils there is a strong perception that samples do get disturbed during the field investigation, transportation and/ retrieval for laboratory testing. There is merit in this argument because, in general, most site investigations are conducted not for research but mainly for actual construction projects. The time or the money allocation for more sophisticated testing is therefore limited by the programme or the budget.

Insitu tests within the Holocene deposits have been carried out using the piezocone (cone penetrometer test with pore pressure measurements, CPTu) and/or Electric Friction Cone (EFC) and insitu vane shear equipment. The industry generally accepts that results from vane shear tests are likely to provide a good indication of the undrained shear strength of the clays and therefore has become a reference test for most projects. The piezocone/EFC test is a probing test and therefore a direct shear strength parameter cannot be measured. However, if calibrated with corrected vane shear data, it provides a continuous strength profile at any test location.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

The common method of converting piezocone/EFC data to obtain a shear strength profile is by using the following empirical equation:

$S_u = (q_t - \sigma_v)/N_{kt}$

where q_t = corrected piezocone resistance,

σ_v = overburden pressure and

N_{kt}= cone factor

Cone factor is found to vary between about 12 and 20 for clays in South East Queensland. Assessment of the field test results at the Port site suggests that an average value of 15 is appropriate as the cone factor for the PoB clays.

The undrained shear strength of the PoB clays is found to increase gradually with depth. Figure 8, reproduced from Ameratunga et al (2005), shows the derived shear strength Vs depth along the eastern part of the FPE Seawall, i.e., the eastern boundary of the FPE reclamation area. The shear strength values have been derived using an N_{kt} factor of 15. To obtain a calibration corrected field vane shear test results were used. The linear variation against depth indicates that the undrained shear strength increases at a rate of about 1.5kPa per metre depth.

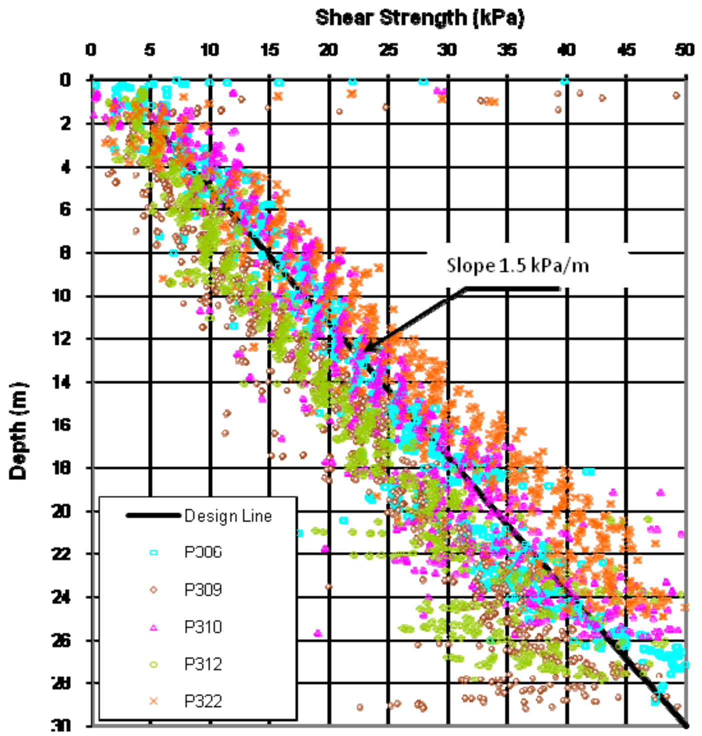


Figure 8 Typical undrained shear strength profile interpreted from a CPTu using a cone factor of 15

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Another important parameter obtained for clays from the vane shear tests is the residual shear strength. The residual shear strength provides guidance on the sensitivity of the clay and the ratio between the peak shear strength, $S_{u(peak)}$, and the residual shear strength, $S_{u(res)}$, is generally known as the sensitivity ratio. The sensitivity ratio values obtained from the tests conducted at the Port are shown in Figure 9 and they indicate that the ratio generally falls between 2 and 4. This confirms the assessments based on liquidity index that the sensitivity of PoB clays can be described as moderate.

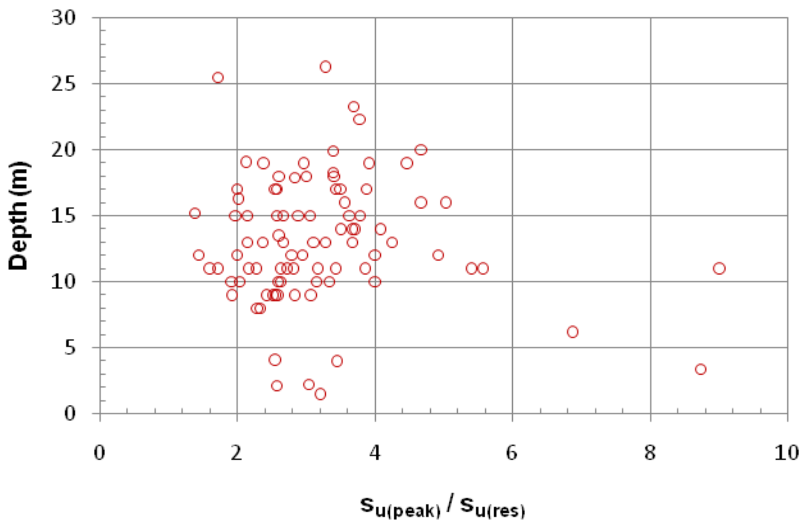


Figure 9 Sensitivity ratio from vane shear test results

Drained Shear Strength of PoB Clay

Although the drained shear strength of a material is of importance in the long term it does not play a major role in the initial stages of construction, specifically where surcharging is the main ground improvement technique adopted. However, the results from the tests conducted for the construction of the FPE Seawall were analysed to assess the long term strength parameters and to understand the behavior of PoB clay characteristics. The results are summarised as q Vs p´ as shown in Figure 10 where

$q = (\sigma_1 - \sigma_2)/2$ $p' = (\sigma_1' + \sigma_2')/2$ and

σ_1 and σ_2 = total vertical and horizontal stress respectively
σ_1' and σ_2' = effective vertical and horizontal stress respectively

After removing the outliers, a best fit line through the origin is shown to give a value of φ'=27.7 degrees. Although a cohesive intercept is evident in some data, considering that the soils are likely to be normally consolidated to slightly overconsolidated, it is prudent to adopt a zero effective cohesion. These values are generally of the same order as for many South East Queensland clayey soils.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

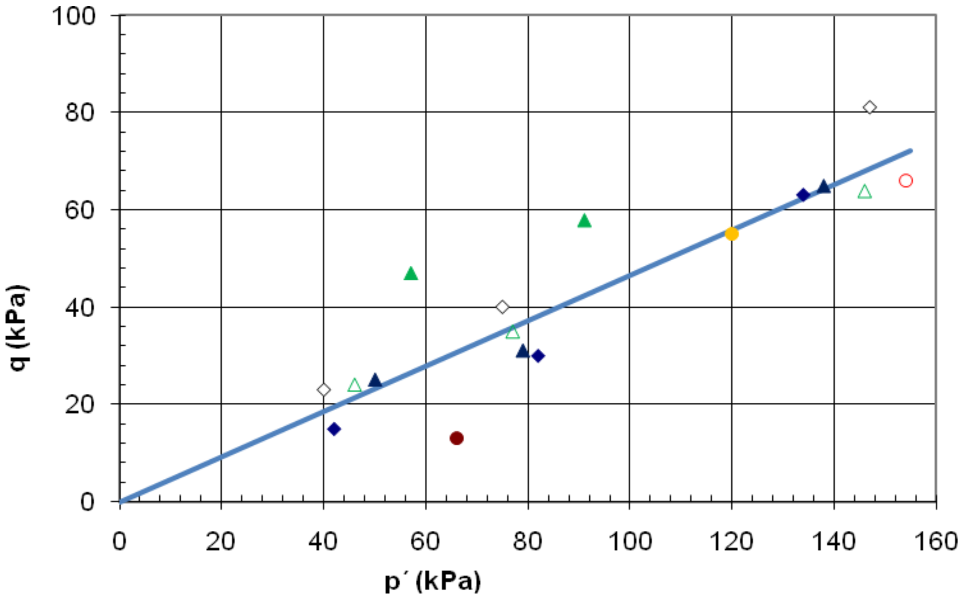


Figure 10 q vs p´ plot for PoB Clay

__Consolidation Parameter CR of PoB Clay__

On a site where the compressible clay thickness is variable and as high as 30m, the magnitude of settlement expected under development loads is high, in the order of 2m to 3m. Ground improvement by surcharging has to be designed to remove the majority of the expected settlement and thus ensure that the long term residual settlement is within limits of performance expected by the PBC, ranging from 150mm to 250mm over a period of 20 years for loads up to 60kPa. Accurate prediction of the consolidation settlement, both primary and secondary, is therefore of key importance.

The Australian geotechnical fraternity still relies heavily on the Terzaghi 1-D consolidation theory and there was no reason not to adopt the same theory for all the project work at the Port. To assess the magnitude of settlements using the Terzaghi theory, two main design parameters are required, viz, CR and $C_{\alpha\varepsilon}$, which are commonly known as Recompression Ratio and Secondary Compression index respectively.

The importance of these parameters necessitated conducting several consolidation tests on samples collected during the field investigations. The majority of the tests used for this assessment were conducted during the ground improvement trials. However, results of a few tests conducted at the site prior to the current trials, and which were available in summary tables and or summary results sheets, were also used in the database.

Case History

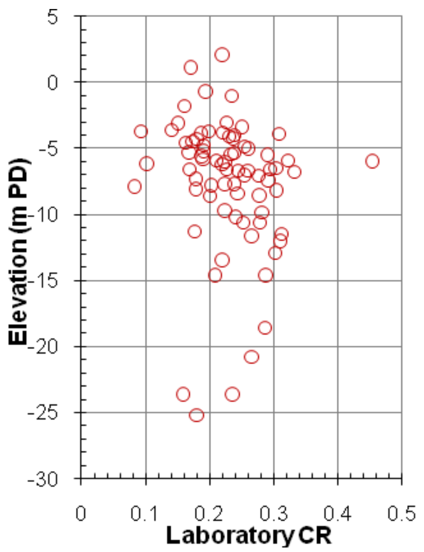
Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

The CR values as obtained from the laboratory e-log p curves for the normally consolidated stress range are plotted against RL in Figure 11(a). The scatter of the data does not appear to be high and CR values generally range between 0.15 and 0.3 with an average value of the order of 0.25. CR can also be obtained from various relationships proposed in published literature linking to physical characteristics such as moisture content and Atterberg limits. Considering that a significant amount of moisture content and liquid limit data is available for the site soils, empirical relationships proposed by the following equations (all relationships taken from Djoenaidi, 1985 who cites the original authors) were used to derive CR values as shown in Figure 11(b) drawn against elevation in PD (Port Datum).

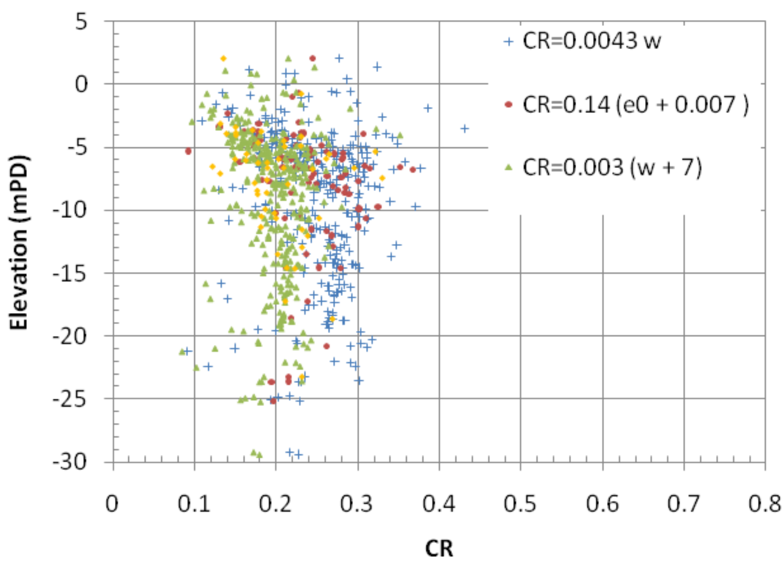
CR = 0.0043 w
CR = 0.14 (e_0 + 0.007)
CR = 0.003 (w + 7)
CR = {0.009 (LL – 10)}/(1 + e_0) (Equation modified from Terzaghi & Peck, 1967 to obtain CR from Compression index)

Where w = natural moisture content
e_0= initial void ratio and LL = Liquid Limit

The range of results appears to confirm the trend shown in Figure 11(a).



(a) Laboratory CR



(b) CR from Correlations

Figure 11 Compression Ratio of PoB Clays

<u>**Consolidation Parameter Cαε for PoB Clay**</u>

Theoretical prediction of settlement is usually confirmed during construction by instrumentation, at least by using simple settlement plates. However, secondary consolidation is almost never monitored in projects because of the long time period required to collect sufficient data. Therefore, designers have almost no information on past projects to refine any measured parameters from laboratory tests. More attention is therefore paid to laboratory tests and correlations based on laboratory tests.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

At the site, as the compressible clay thickness is large, the secondary consolidation component is a significant amount to be dealt with. Therefore it is important that the surcharge design takes into account the likely secondary consolidation and removes some of the settlement by having a higher surcharge. When the subsoils are subjected to a higher load than the expected development design loads, the clays will become overconsolidated (OC). The amount of over consolidation to be targetted in the design is a difficult question to the designer because of the uncertainties in secondary consolidation behavior. In the past, many researchers have shown that the secondary consolidation expected from an overconsolidated clay is significantly lower than that expected when the same layer is in a normally consolidated (NC) state. Generally, the industry believes that a drop to 1/5 or 1/10 the value of NC state can be expected if a clay layer is surcharged above its normally consolidated state. Several researchers have demonstrated that $C_{\alpha\varepsilon}$ is dependent on the amount of over consolidation achieved, which is generally expressed as OCR (over consolidation ratio). OCR is calculated by dividing the final effective stress under the surcharge load by the effective stress expected under the stresses imposed by the expected design loads. Figure 12 shows available data from several researchers and cited by Wong (2007) plotted as a Creep Ratio of $C_{\alpha\varepsilon\ OC}$/ $C_{\alpha\varepsilon\ NC}$ against OCR. Results available from the PoB site have also been plotted on the same figure, which appear to show that the Creep Ratio drops very quickly with a small increase in OCR. Further testing would be required, especially at low OCR values in the range 1.0 to 1.2, to confirm these findings. These results suggest that, if clays in the field behave in a similar manner to that in the laboratory and surcharging can achieve at least a nominal OCR, the risk of high long term settlement due to secondary consolidation can be reduced significantly.

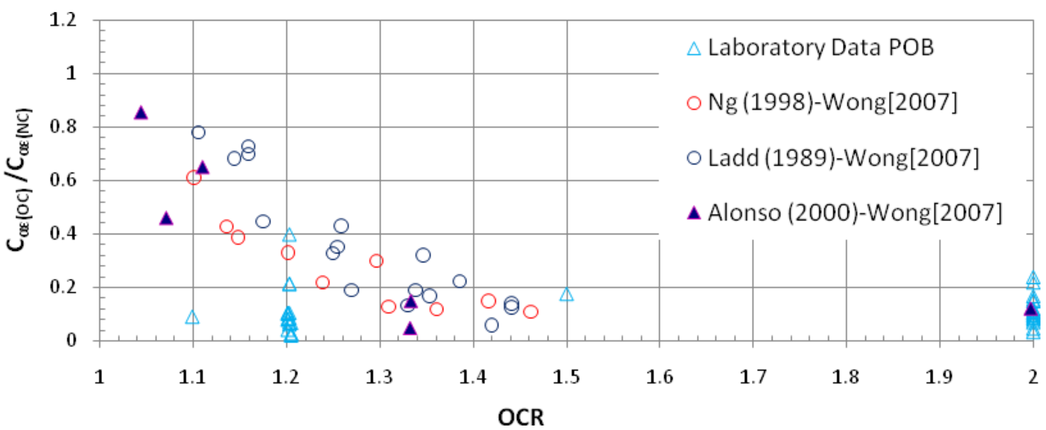


Figure 12 Creep ratio Vs OCR

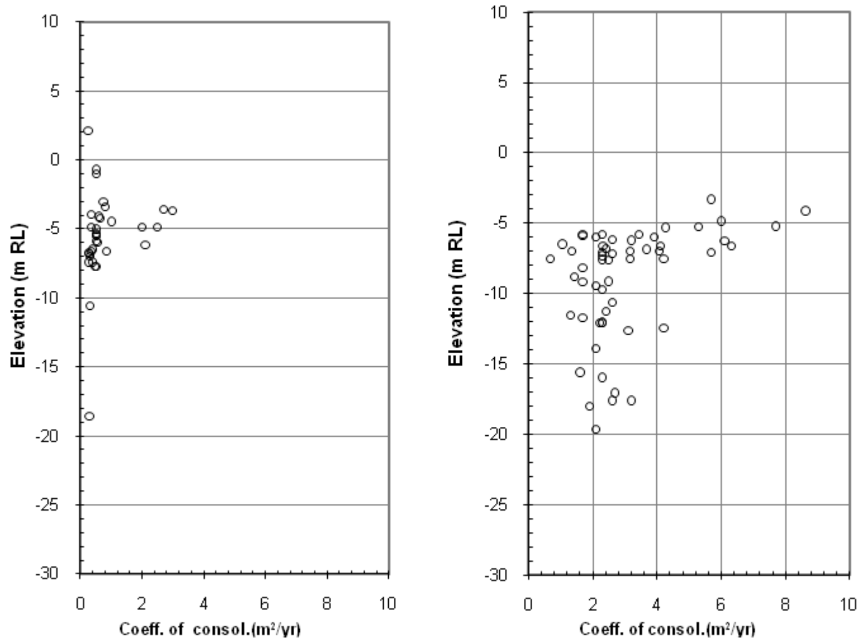
<u>**Coefficient of Consolidation of PoB Clay**</u>

The settlement rate is governed by the coefficient of consolidation (c_v) and it is one of the most difficult parameters to assess from routine testing. In South East Queensland, generally the laboratory consolidation tests provide values that are too low when compared to back-calculated values from field monitoring. Field values of 5 to 10 times the laboratory values are not uncommon (Lambe and Whitman, 1969). In South East Queensland, more attention is given to values derived from piezocone dissipation tests than from laboratory test results. Generally a dissipation test is conducted in a layer identified from information available at the site and is important to the designer. The best method of identifying a layer is by carrying out a piezocone test at the location of interest which provides a full profile of the subsoil strata. Once the test depths are identified the piezocone is pushed again slightly offset from the original location and dissipation tests are carried out. Carrying out dissipation tests by interrupting a normal probing test is not recommended because layer identification is not possible before the test.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Figure 13(a) shows the results of c_v obtained from laboratory tests on samples of the PoB clays. The results indicate an average value less than 0.5m^2/yr. The results of the piezocone tests across the same site are shown in Figure 13(b). We have assumed that a piezocone dissipation test provides a horizontal coefficient of consolidation (c_h) rather than a c_v value but this is debatable because of the complex conditions associated with pore pressure dissipation. The piezocone test results in Figure 13(b) show that the insitu value is 4 to 8 times higher than the laboratory values from consolidation tests. Back-calculation of trial results appear to suggest that c_h obtained from piezocone tests is of the right order, assuming a horizontal to vertical ratio (c_h/c_v) of 2. Therefore piezocone tests are considered to offer better assessments of the coefficient of consolidation than the laboratory tests for the PoB clays and this seems to confirm the general trend in South East Queensland.



(a) Laboratory tests (b) Piezocone tests

Figure 13 Coefficient of consolidation

Wick Drain Performance in South East Queensland, Australia

Case histories of wick drains are widely available in geotechnical literature and the reader is referred especially to the works of Brand and Brenner, 1981, Balasubramaniam et al, 1984, Bergado et al, 1996, Indraratna and Chu, 2005. Wick drains and sand drains have been used to improve soft clays in conjunction with preloading/surcharging over many years in Australia and overseas. While there are many case histories presented in conferences and seminars, only success stories are generally available in the public domain because many do not wish to discuss failures and even if they want to, because of commercial and/or legal consequences, they practically cannot.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Compared to three decades back, more research has been conducted on wick drains and sand drains in the laboratory as well as calibrating field observations in trial embankments and actual projects. The most famous ones are the works associated with Bangkok airports (see Balasubramaniam, 2010) and the Malaysian trial embankments (Poulos et al, 1989; Nakase and Takemura, 1989; Indraratna et al, 1992; Brand and Premchitt, 1989). With more knowledge gained from these projects and research work, improvements on machinery with vibration not currently used for pushing a mandrel, and increased expertise of installers, it is reasonable to accept that the performance of wick drain installations would be better than in the past. Just because the technology is robust, it does not mean the use of wick drains is without risk because the performance of a wick drain still depends on various factors, especially smear, which are directly related to the subsoil characteristics. Hence the issue of wick drain performance still challenges geotechnical professionals, even the leading academics, researchers and practitioners.

In South East Queensland, especially along the coastline, the occurrence of soft soils is widespread and the use of wick drains in conjunction with surcharging has been prevalent. Acceptance of wick drains as a genuine ground settlement acceleration technique by the industry has been slow because of anecdotal evidence, as well as reported cases where wick drains have underperformed or not performed at all.

Robertson (1984) reported a case of preloading in the Oyster Creek vicinity where an estuarine depositional environment existed and the Pacific Highway approach embankments were up to 9m high. The subsurface profile consisted of 3m to 5m of silty sand overlying up to 16m of soft organic clay. The solution adopted was surcharging with wick drains spaced at 1.7m and 1.9m triangular. However, a closer spacing of 0.8m was adopted under the culvert area (three cells of 1800mm pipes) to achieve early completion. Robertson concluded that, overall, wick drains performed well except for the closely spaced wick drains in the culvert area. The more demanding time of 12 months was not satisfied and the target settlement was not achieved until 20 to 22 months after embankment construction. The author ascribed this slower rate of consolidation to most probably the disturbance caused by a closed mandrel 150 x 75 mm driven at 0.8m centres.

Wijeyakulasuriya et al (1999) described the results of trial embankments on soft sensitive clays constructed along the eastern coast, Sunshine Motorway and the Gold Coast. The undrained shear strength of the clays was around 10 to 15kPa, with natural moisture contents generally between 60% and 120%, and a liquidity index ranging from 1.5 to 2.5. These characteristics suggest the clays to be highly sensitive to disturbance. The authors also stated that piezocone dissipation tests were masked by the remoulding of the clay caused by insertion of the cone. At the Sunshine Motorway site the Motorway traversed a swamp underlain by very soft to soft organic marine silty clay, ranging in thickness from 4m to 10m. The trial embankment comprised three 20m sections; Section A had wick drains of 1m spacing, Section B had wick drains with 2m spacing, and the middle Section acted as a Control Section. Comparison of results of settlement plates indicated that wick drains had not accelerated settlements significantly, with the 2m section settling the most. Wijeyakulasuriya et al (1999) concluded that the results suggested that the advantage of closer spacing of the wick drains had probably been almost wiped out by installation disturbance of the wicks in these sensitive deposits. They recommended that if ground improvement techniques such as wick drains were to be considered in sensitive soils, careful consideration should be given to the geotechnical conditions because of the potential for underperformance of the wick drains.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

PBC also carried out a small trial towards the west of the current reclamation site about 10 years back. The trial was conducted in an area located at the Berth 8 Extension, i.e., about 1 km west of the current site. As seen from Figure 14, the site is underlain by about 10m of compressible clay. The trial consisted of wick drain areas of different spacings, one 0.8m diameter sand drain area and a control area. The analysed results (Coffey 2004) suggested that wick drain areas with 1.2m and 1.6m square spacings did not perform at all compared to the control area whereas the use of 0.8m diameter sand drains installed using vibro-replacement techniques appeared to produce an accelerated rate of consolidation at the early stages of loading. A closer examination and analysis of the results (Figure 14) indicates that the settlement rate was significantly quicker for the sand drain area than the control area, but subsequently, the settlement rate per log (time) cycle was practically identical. The settlement separation between the two plots is only about 50mm which could be related to the upper layers interbedded with sand. The fact that wick drains did not appear to have performed (in fact slightly lower rate of consolidation compared to control area) suggested, among other things, that the effect of smearing may have been significant and suggested that steps must be taken in the installation method to reduce disturbance if wick drains are to be considered. It is now generally accepted that vibratory methods should be avoided in the installation of wick drains, especially where the soils are sensitive.

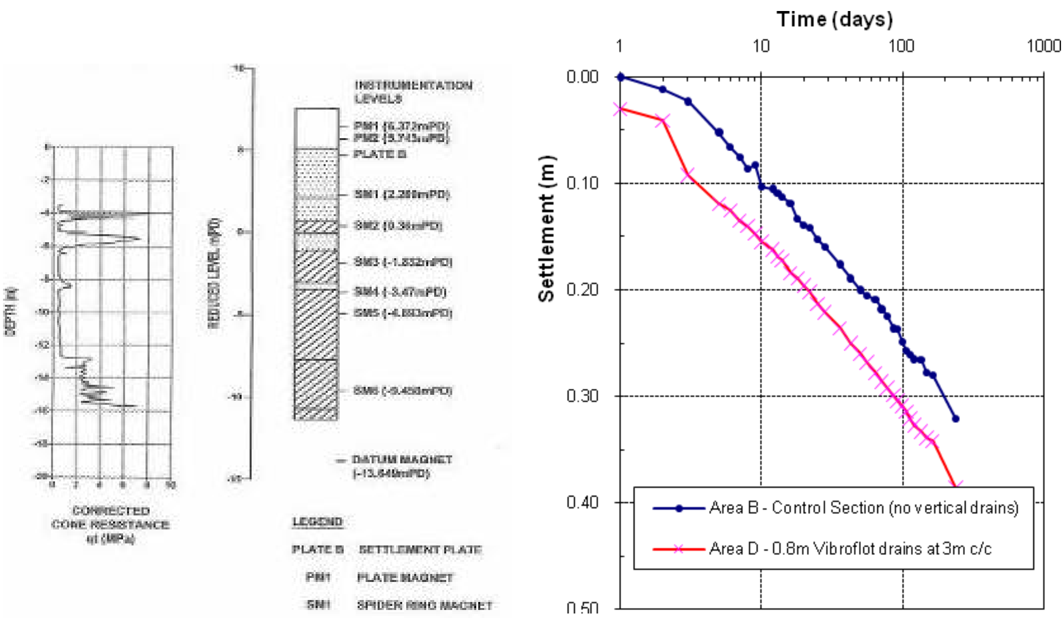


Figure 14 Trial embankment at the Port of Brisbane in 2000

Port of Brisbane – Ground Improvement Trials

PBC and Coffey carried out in-house studies to assess the best course of action to move forward and develop the land available for development. As previously discussed, the clay thickness significantly increases towards the northeast in addition to the increased thickness of dredged materials that could be held in the paddocks created and contained by the seawall and internal bunds. Therefore, conventional techniques such as surcharging without wick drains would not be feasible if the land is to be developed over the next 25 years or so along commercial timelines.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Internal studies carried out by Coffey and the PBC into local and overseas practices of treatment of soft soil indicated that, apart from conventional surcharging, the two main groupings of techniques available to treat and improve the reclamation spoil and in-situ soils were:

1. Improve, reinforce or stabilise the soils to reduce settlements and to improve shear strength and stiffness
2. Surcharging in conjunction with accelerating settlements.

The suite of techniques falling under group 1 includes stone columns, piling of the ground, deep soil mixing and mass stabilisation. Most of these techniques needed large scale equipment to deal with deep soils on site and therefore the mobilisation costs for a small trial were excessive, as the equipment had to be brought either from Asia or Europe. The methods falling into group 2 comprise vertical drains and sand drains. Vacuum consolidation, which is a process whereby a vacuum pressure is applied to an area already installed with wick drains to potentially increase their effectiveness, also falls into this category.

The most economical consolidation technique identified was to use prefabricated vertical drains, commonly known as 'wick drains', in combination with surcharging, to accelerate the consolidation process. However, PBC was concerned about wick drain underperformance. As it ultimately needs to treat a reclaimed area in excess of 235ha, PBC needed some certainty in relation to the performance and timelines for consolidation and therefore decided to conduct large scale wick drain and vacuum trials to assess the effectiveness and performance of the various techniques. This would enable optimised designs to be used in future treatment of the large areas of the reclamation site.

PBC invited Expressions of Interest (EOI) from Australian and overseas specialist ground improvement contractors for the design, supply, installation and monitoring of applicable specialist ground improvement systems. Although available information on subsoil profiles and preliminary design parameters were provided in the EOI documentation, no recommendations were given on the type of technique to be used and it was left to the potential tenderers to propose what was best for the site. The EOI did permit any solution and specifically requested suggestions for improvement techniques for the edges bounding the Moreton Bay Marine Park, because it was critical to avoid disturbance to the Marine Park and nearby internationally recognised Ramsar site for migratory wader birds.

PBC and Coffey set up selection criteria to assess the eight tenders received based on solution, design, price, quality systems and environmental considerations. All proposals were assessed by scoring them against the selection criteria. This resulted in the short listing of three preferred proposals. These three submissions could not be substantially separated in terms of the selection criteria, with all three offering wick drain solutions. Vacuum consolidation was also proposed, mainly for the edge boundary conditions.
PBC decided that there was considerable merit in trialling all three contractors rather than further reducing the number of trials. The three successful tenderers were Austress Menard (Menard), Boskalis and Van Oord. To optimise the designs, a range of drains were proposed for trialling, with different spacings, patterns and filter fabrics. Menard's vacuum consolidation system was more applicable where slope stability was an issue, such as the treatment of the edges of sites, as the vacuum pressure applied reduces the amount of surcharge loading required, thus improving edge stability. The applied vacuum pressure was about 80kPa, which is equivalent to about 4m of sand surcharge fill. Therefore it was selected for a trial site which posed significant stability risks on two sides as it is bounded by the Moreton Bay Marine Park and the Port's purpose built Migratory Wader Bird Roost, which has a perimeter moat around it to protect the birds from predators. A unique feature of the vacuum system adopted for this latter site was the 15m deep soil-bentonite cut-off wall which was required to isolate the vacuum area because of the deep, permeable, sand lenses and layers within the subsurface profile. Appropriate testing was carried out to assess the depth of sand lenses and properties of the overburden prior to excavation of the trench. This is the first occasion that vacuum consolidation with a cut-off was ever used in Australia.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

The areas set aside for the trials were expanded and rationalised to allow different types of trials to be conducted. The trial types varied from the wick drain type, filter type, spacing and configuration. Boskalis also trialled its BeauDrain vacuum system in a small area. Coffey, in consultation with the trialists, carried out detailed in-situ and laboratory investigation for all areas and provided the results to the trialists.

Figure 15 shows the trial areas and Table 2 provides the main information related to those areas. Three specialist reviewers, Profs Harry Poulos from Coffey Sydney office, Prof A S Balasubramaniam from the Griffith University and Prof Buddhima Indraratna from the University of Wollongong were appointed by the PBC to act independently as Specialist Reviewers of trial performance.

Extensive instrumentation was installed by Coffey and the PBC in consultation with the trialists. Instruments were regularly monitored during and after construction and the results were made available via an on-line data management system 'Insite', specifically modified by the supplier to cater to the needs of the trials. The Insite system enabled monitoring results to be viewed by the relevant trialists and others such as Coffey and Specialist Reviewers. Interim reviews of the results and back-analyses were performed by the trialists, checked by Coffey and the performance independently assessed by the Specialist Reviewers.

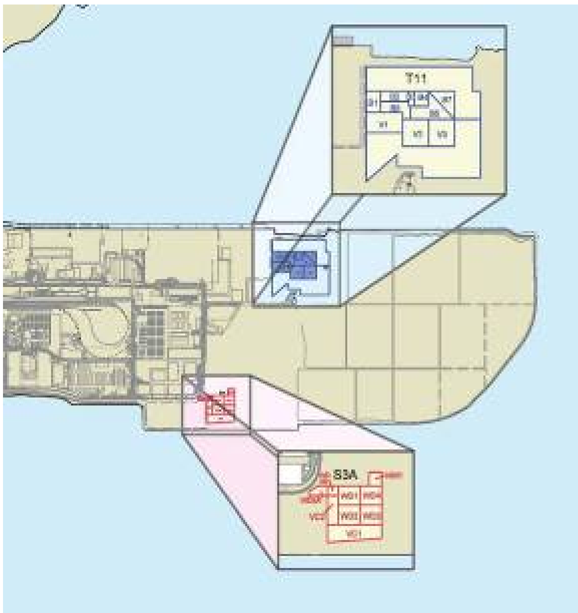


Figure 15 - Trial areas

Table 2 Details of the Trials

	Boskalis	Van Oord	Austress Menard (wick)	Austress Menard (vacuum+wick)
Design load	60kPa	60kPa	15kPa	15kPa
Trial size	30,000m^2	30,000m^2	27,210m^2	16,380m^2
Wick types	MD7007, MD88H, MD88HD	MD7007, MD88H	MD88, FD767, MCD34	MCD34
Wick grid	Triangular	Triangular	Square	Square
Wick spacing	1.25m	1m - 1.4m	1.1m - 1.3m	1.2m
Design Preload period	12 months	6-12 months	7 - 12 months	8 months
Surcharge height	5 - 9m	7.5 - 10m	3 - 8m	2.5m

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Technical Objectives of the Trials

The purpose of the project was to acquire knowledge on effective techniques to accelerate the process of reclaimed land development at the Port of Brisbane and to optimise the treatment solutions for the varying ground conditions encountered across the Future Port Expansion Precinct:

- achieve design loads ranging from 15kPa up to 60kPa;
- reach a post-development residual settlement of 150 mm to 250mm maximum over 20 years under service loadings;
- complete consolidation within 12 to 18 months maximum;
- identify contributions to overall settlement made by the various compressible layers of dredged mud, upper Holocene and lower Holocene;
- develop a range of optimal PVD solutions applicable to the variable conditions that occur at the site;
- identify the most efficient and cost effective PVDs best suited to the varying ground conditions;
- optimise the installation procedures and relative spacing of the PVDs to ensure that excessive surcharging is not required;
- identify designs that promote low risk edge stability of sites to be surcharged;
- establish future strategies, options and trials for the deeper and more complex areas of the reclamation area; and
- develop long term strategies involving the consolidation of dredged mud, to limit the use of sand for fill and surcharging.

Results and Back Analyses

The monitoring results were available to the contractors, Specialist Reviewers and Coffey. Each party used different techniques to back-calculate design parameters as well as predict future residual settlements. Coffey adopted the following methodology:

1. Set up a geotechnical model and filling history for each assessment location.
2. Derive time settlement curves using its in-house software package CAOS for all monitoring locations, based on design parameters adopted after the sensitivity analysis for the trial area in Paddock S3A. This prediction takes into account the actual fill thickness, thickness of compressible units, fill construction history etc. In areas where extensometers have been installed, assess the compression of individual layers to avoid the uncertainties associated with the multi-layered system and thick soft soil issue.
3. Assess the curve fitting and adjust the fill history and loading where appropriate. The analyses have shown that the settlement predictions are mostly affected by fill history and load applied.
4. Review results using the Asaoka method (Asaoka, 1978) as an additional tool. The corresponding pore pressure data from piezometer monitoring was reviewed for consistency.

The inhouse software package CAOS (Consolidation analysis of Soft soils) is a FORTRAN program that carries out a finite difference numerical solution of the one-dimensional equation of consolidation (Poulos, 2002). It can consider a multi-layered soil profile subjected to a series of loading sequences which may include both constant and time-depending loading. In the analyses, the following have been taken into consideration:

- Buoyancy effect of the fill below the groundwater level due to settlement;
- Design limit on post-construction settlement;

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

- Long-term creep effects.
- Effect of smear due to mandrel insertion

The output from the program includes the surface settlement, the distribution of settlement and excess pore pressure with depth at specified times, and the soil state (i.e. normally- or over-consolidated state) at various depths in the profile.

The theory developed by Hansbo (1981) has been utilised in CAOS. The effects of smear during the insertion of mandrel have been also incorporated in the programme. The effect of smear can be controlled by inputting the radius of smear zone and ratio of permeability of undisturbed soil to that of remoulded soil. The time of installation of the vertical drains can also be specified.

The program allows the user to select one of two options for computing the settlements due to creep:

1. A method based on Bjerrum's (1967) concept of instantaneous and delayed consolidation;
2. A method in which creep is initiated when the percentage dissipation of excess pore pressure reaches a value specified by the user. This approach was adopted.

Furthermore the post-construction settlements have been based on the concept that the rate of creep reducing with increasing over-consolidation ratio (OCR) (Mesri et al., 1994) as described in Wong (2006). An exponential law for the reduction of $C_{\alpha\varepsilon}$ with OCR has been adopted and given below.

Creep Ratio = $\dfrac{C_{\alpha\varepsilon(o/c)}}{C_{\alpha\varepsilon(n/c)}} = \dfrac{1-m}{e^{(OCR-1)n}} + m$ where "m" and "n" are constants.

Constant "m" represents the minimum values of Creep Ratio when the OCR is large. From Mesri (1991) "m" will be equivalent to the ratio between recompression and compression index (C_r/C_c). The magnitude of "n" controls the rate of reduction of the Creep ratio with OCR. The adopted values for the project were *m = 0.1 and n = 6.* If the Creep Ratio is plotted against OCR based on these selected values, the resulting curve is shown in Figure 16 with laboratory Creep Ratio values reproduced from Figure 12 in the background. The adopted equation is considered reasonable and conservative.

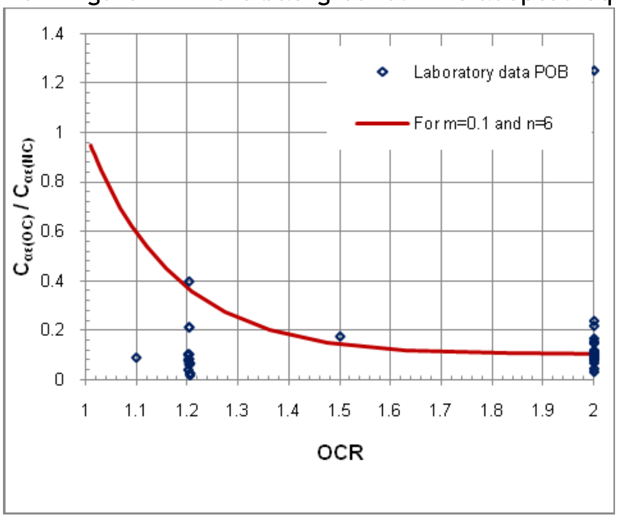


Figure 16 Creep ratio Vs OCR

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Due to logistical difficulties not associated with the contractors, it was not possible to begin all trials at the same time. Menard vacuum and wick drain trial area was the first to commence and generally was not affected by a temporary shortage of fill for surcharging operations as for the other trials. This assisted the Menard trial to be completed according to the programme envisaged. This also provided all parties, especially the three Specialist Reviewers, with the opportunity to analyse the data in a more detailed manner. Therefore the Menard vacuum trial was selected in this paper to highlight some of the more interesting lessons learnt through the trial process before summarising other findings.

Menard Vacuum and Wick Drain Trial

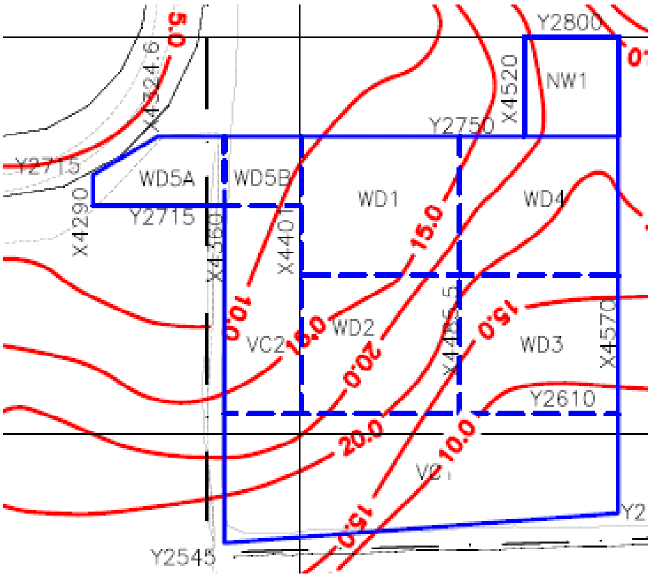
The Menard vacuum trial was located on the southern side of the reclamation in Paddock S3A as shown on the plan in Figure 15. An aerial view of the trial area taken when the membrane has been installed is shown in Figure 17.

Figure 17 Menard Trial-Vacuum area with membrane placed and adjacent wick drain area

The subsurface profile across the Menard trial area can also be described as recent fill overlying Holocene deposits which in turn overlie the stiffer Pleistocene deposits and the basalt bedrock. The recent fill consists of the dredged mud across the paddock which is variably thick, overlain by a sand capping mainly constructed to facilitate the movement of construction vehicles. As the dredged mud is very soft, not more than 5kPa in undrained shear strength, it was necessary to have a minimum thickness of sand of 2m with localised thickening where necessary. During placement of the sand capping in March 2006, the dredged mud in the reclamation paddocks displaced significantly (mud waving), giving rise to a variable distribution of the mud and sand cap across the trial site. The sand layer also acted as the drainage layer for the wick drain operations. The thickness of the Holocene layer is also variable as demonstrated by the clay thickness contours shown in Figure 18. As previously discussed, the upper Holocene layer consists of mainly sand with inter-layered soft clays and is therefore highly permeable with the lower layer, comprising very soft to firm clay, being generally normally to slightly consolidated and therefore highly compressible. A summary of soil strata is given in Table 3.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)



VC1 and VC2 - Vacuum + wick drain areas

WD1 to WD5 - Wick drain areas

NW1 - Control area

Figure 18 - Lower Holocene clay thickness (in metres) contours across the Menard trial area

Table 3 Geological units

Unit	Description	Thickness Range
Recent - Sand capping	White sand	0.9m~5.0m
Recent - Dredged materials	Dredged mud, marine and dune sands with layers of silt and clay.	2.5m~7.6m
Holocene	Marine clay, silt and sand.	8.8m ~24.4m
Pleistocene	Generally stiff clay, sand and gravel	
Tertiary	Bedrock	

The Menard Vacuum Consolidation method consists of installing vertical and horizontal vacuum transmission pipes under an airtight membrane and sucking the air below the membrane thus imposing a partial atmospheric pressure on the soil, creating an accelerated isotropic consolidation; which can be combined with a conventional surcharge placed on top of the membrane, in order to achieve the required degree of consolidation (See Figure 19) (Berthier et al, 2009). A critical element of the method as applied to the Port site was the construction of a deep soil bentonite cut-off wall, to isolate the site from the surroundings because of the frequent sand lenses and layers observed in the upper Holocene layers. This seal is crucial to the efficient functioning of the vacuum method. It is in fact the first occasion the vacuum consolidation with a cut-off wall was ever adopted in Australia. The cut-off wall depth was 15m, the deepest ever employed with the Menard Vacuum method.

As detailed in Berthier et al (2009), a vacuum pressure of 80kPa was maintained throughout the trial period as measured in piezometers embedded in selected wick drains at deep levels, and in vacuum gauges placed under the membrane. This vacuum is equivalent to about 4m of fill surcharge in a routine surcharging scenario. Therefore only an additional surcharge of up to 2.5 m was required for the vacuum area of the trials.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

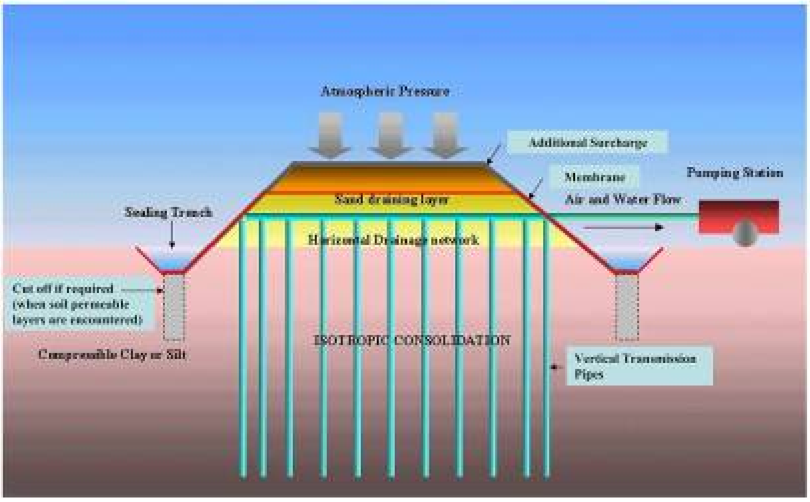


Figure 19 Menard vacuum consolidation method (after Berthier et al, 2009)

As shown in Figure 17, the non-vacuum wick drain trial sites of Menard were adjacent to the vacuum areas and consisted of one Mebradrain type, one Flexidrain type, and a circular drain 34mm diameter. Spacings ranging from 1.1m to 1.3m square were adopted for the Menard wick drain trials.

As Balasubramaniam et al (2010) discuss, Asaoka method (1978) and hyperbolic models are the most widely used methods by engineers, with Asaoka method most popular within the Australian industry. These methods are most useful in estimating the percentage of consolidation under the surcharge. It has its limitations mainly because it is applicable only to a single layer of homogeneous clay. This in itself is not a major issue if one of the layers is significantly thick relative to the other and only the latter stages of consolidation are of concern. Thus, it is generally accepted that the Asaoka method would be applicable only in the latter part of the consolidation curve, viz., after at least 60% of consolidation is completed. However, if it is required to interpret the behavior from an early stage, especially when time is limited and any remedial measures need to be implemented earlier, other methods need to be adopted such as the CAOS software previously discussed. Even such software would not eliminate the possibility of deviations in the latter part of the curve, but they allow sensitivity studies to be conducted and perhaps provide the engineer useful guidance necessary to take intervention measures early. In the current trials, both Asaoka and curve fitting techniques were adopted. While Coffey used the CAOS software, other trialists and the Specialist reviewers used their own software (e.g. Indraratna, 2010) and/or commercially available software such as MSettle.

Figure 20 shows a typical settlement curve measured in the field and a calibrated curve in the vacuum area (Berthier et al, 2009). Calibrations were carried out by trial and error but using knowledge gained on parameters from laboratory tests and/or empirical knowledge to select which parameters were likely to be variable. In complex sites such as that at the Port of Brisbane, where the variables are numerous because of the layering of the soils, there is the potential that several solutions could provide similar answers.

The comparison of vacuum and non-vacuum provided interesting observations as illustrated by the average Degree of Consolidation (DOC) plots in Figure 21 (Indraratna, 2010a). It is quite clear that the rate of settlement under vacuum (VC1 and VC2 areas) during the initial stages is greater than in the non-vacuum area. Although the two curves should merge at the end of primary consolidation, accelerated rates of settlement in the early part provide a significant advantage especially where stability is a key issue of concern.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

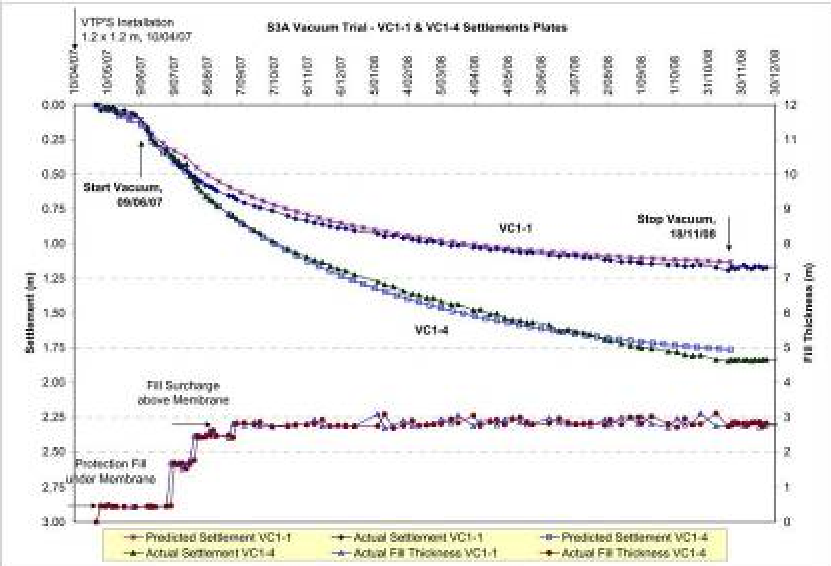


Figure 20 – Typical settlement curve in the Menard vacuum area

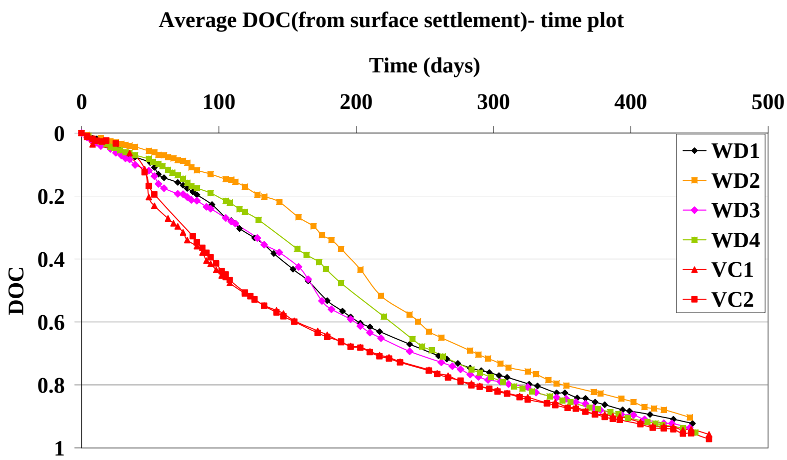


Figure 21 – Time settlement curves for vacuum and non-vacuum areas

The stability during construction was assessed by monitoring the lateral movement by inclinometers as well as measuring the excess pore pressure behavior using piezometers. The primary reason a vacuum solution was adopted at the edges was the high risk of instability under the surcharging operations. The lateral movements from the inclinometers were studied in detail by the Specialist Reviewers to assess whether the vacuum system provided a significant advantage. Figure 22 (Indraratna, 2010a) shows the lateral displacements at two inclinometers, one at the vacuum area and the other at the non-vacuum area. Indraratna has removed the difference in the applied stress at the two locations by "normalising" using the effective stress. The reduction in lateral movement is clearly shown in the figure confirming the advantage the vacuum system offers. The benefits of the vacuum in controlling the lateral movement is further discussed by Indraratna (2010b) in his 2009 E H Davis Memorial lecture.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

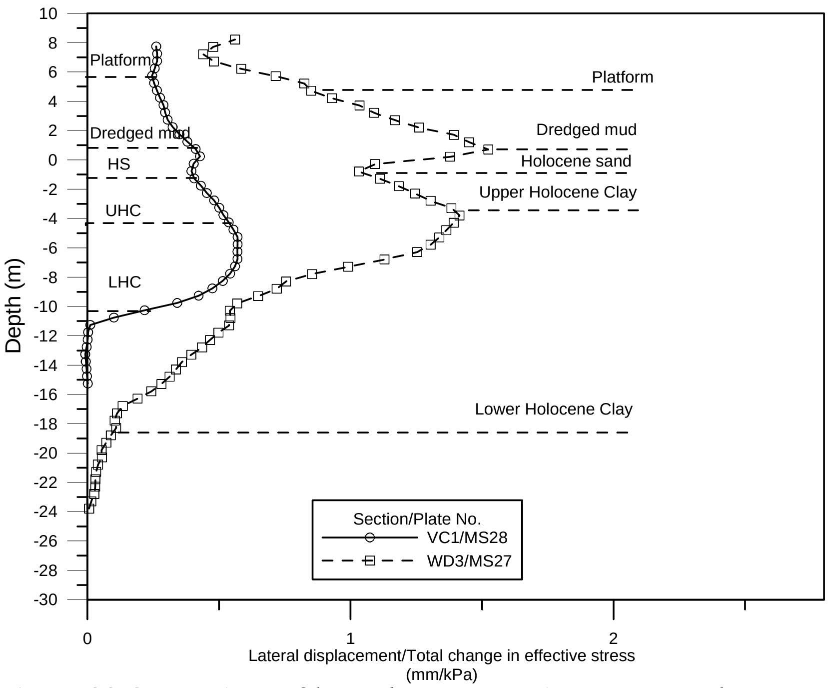


Figure 22 Comparison of lateral movement in vacuum and non-vacuum areas

Learnings from the Trials

Some of the learnings from all the trials are summarised below.

- All flat and round drain trials worked satisfactory. The following drains and filters were trialled successfully: MD88*, MD88H, MD88HD, MCD34, FD767, MD7007 and MD88 drains. None of the drain cores indicated unusual behavior such as breaking, kinking etc although significant strains were experienced by the drains. MD88 showed a slightly better performance in the degree of consolidation achieved but the difference of 5 to 10% is insufficient to make a concrete conclusion.
- All the drains trialled had filters; there was no marked difference when a larger filter pore size was used, indicating that the pore size is not critical, at least for the conditions at the Port (the filter pore sizes of 75µm, 80µm and 150 µm were trialled); It was suggested that a pore size close to 75µm be used in the future as it indicated a slightly better performance;
- Spacings from 1m to 1.4m were trialled and a definite reduction in performance was observed for the closer 1m triangular spacing. This was assessed to be due to the greater effect of smear when the drains were closely spaced. The advantage of closer spacing seems to be negated by the increased effect of smear.
- In one area of the Port, wick drains were installed but the surcharging was delayed for 18 months. The drains continuously worked under the load from the sand capping but only low volumes of water were discharged under this surcharge. The performance of these drains under the full surcharge suggests that leaving the wick drains in the ground over such a long period after installation, does not diminish the performance of the drain providing the ground is settling i.e. as water is discharging through the drains due to consolidation caused by loading such as sand capping. If there is no water discharge, there may be possibilities of blocking of pores caused by clay particles.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

- Vacuum consolidation, although relatively expensive, is a better performing system compared to wicks and surcharge alone. The cut-off wall provided a very effective seal. There is a much faster strength gain in the early stages because of quicker dissipation of pore water pressure, compared to wicks only, providing a significant advantage in terms of lateral stability.
- Vacuum consolidation clearly shows accelerated settlement rates in the initial critical stages of surcharge performance compared to non-vacuum trials. This is most helpful in relation to stability.
- Vacuum consolidation also shows that the lateral movement is clearly reduced by the application of the vacuum and the less surcharge heights involved. This emphasises its advantage in controlling stability.
- The round drain worked well in the vacuum consolidation areas, but there was no advantage observed for other areas.

Concluding Comments

A series of ground improvement trials using wick drains and vacuum consolidation was carried out at the Port of Brisbane because of concerns of underperformance of wick drains in previous projects in South East Queensland. The purpose of the project was to ascertain whether wick drains would perform efficiently under the conditions at the Port and to acquire knowledge on effective techniques to accelerate the process of reclaimed land development and to optimise the treatment solutions for the varying ground conditions encountered across the Future Port Expansion Precinct.

Although field and laboratory tests of samples undertaken previously provided valuable assessment of the design parameters, they cannot replicate the full-scale field testing of the in-situ soil conditions undertaken in these trials.

The trials have provided PBC with confidence that wick drains can be adopted to treat the deep soft clays at the Port of Brisbane. Excessive smearing can reduce the performance of wick drains significantly, although this may not be a major issue at the Port site, probably because the sensitivity of PoB clays is not as high as in some of the soft clays found elsewhere in South East Queensland. Nevertheless, the trials showed that too close a spacing could lead to some reduction in performance, due the effects of smearing.

The Menard vacuum consolidation trials clearly demonstrated that vacuum consolidation application for PoB clays does work effectively and that excellent results can be obtained once cut-off walls are adopted to isolate the sand lenses in the upper horizon (upper Holocene deposits) from the wider area.

The trials emphasise the need to conduct full scale trials in major projects whenever possible. They are better than any other tests, insitu or laboratory, and provide valuable information to the designer which ultimately will assist the client in reducing his risk and the cost of completion of a project.

Acknowledgements

The authors wish to acknowledge the following:

- Dr Bill Tranberg and Arie Geelhoed of the Port of Brisbane Corporation
- The Trialists, Austress Menard, Boskalis and Van Oord
- Ms Tine Birkemose, Ms Chamari Bamunawita, Dr Kamal Dissanayake, Mr Thayalan Nallarulanantham and Mr Zen Ng of Coffey Brisbane, and Mr Patrick Wong of Coffey Sydney
- The three Specialist Reviewers Prof Harry Poulos, Prof A S Balasubramaniam and Prof Buddhima Indraratna and
- Prof Harry Poulos for reviewing this paper.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Port of Brisbane in 2009 (Courtesy of Port of Brisbane Corporation)

References

Ameratunga, J., Boyle, P., De Bok, C. and Bamunawita, C. (2010). Port of Brisbane (PoB) clay characteristics and use of wick drains to improve deep soft clay deposits. Proc. 17th Southeast Asian Geotechnical Society Conf., Taipei, Vol I, pp 116-119.

Ameratunga, J., Shaw, P., Beohm, W. and Boyle, P. (2005). Seawall construction in the Moreton Bay, Brisbane. Proc. 16th Intl Conf. on Soil Mechanics and Geotechnical Engineering, Osaka, pp1439-1442.

Asaoka, A. (1978). Observation procedure of settlement prediction. Soils and Foundations, Volume 18, Issue 4, pp87-101.

Balasubramaniam, A.S., Cai, H., Zhu, D., Surarak, C. and Oh, O. Y. N. (2010). Settlement of embankments in soft soils. Proc.17th Southeast Asian Geotechnical Society Conf., Taipei, Vol II, pp 31-49.

Berthier, D., Boyle, P., Ameratunga, J., De Bok, C. and Philippe, V. (2009). A successful trial of vacuum consolidation at the Port of Brisbane. PIANC 2009, Wellington.

Brand, E.W. and Brenner, R.P. (1981). Soft clay engineering. Elsevier Publications.

Brand, E.W. and Premchitt, J. (1989). Moderator's report for the predicted performance of the Muar test embankment. Proc. Trial Embankments on Malaysian Marine Clays, Volume 2, The Malaysian Highway Authority.

Coffey Geosciences Pty Ltd (2004). Onshore reclamation study – Paddocks S2, S3a and S3b. Report B17509/9-H submitted to the Port of Brisbane Corporation.

Djoenaidi, W.J. (1985). A compendium of soil properties and correlations. M.Eng Sc Thesis, University of Sydney, Australia.

Case History

Ground Improvement in Port of Brisbane (PoB) Clay (cont.)

Hansbo, S. (1981). Consolidation of fine-grained soils by prefabricated drains. Proc. 10th Intl Conf. SMFE, Stockholm, 3: 12-22.

Indraratna, B. (2010a). Personal Communication.

Indraratna, B. (2010b). Advances in the application of vertical drains and vacuum preloading in soft soil stabilization. 2009 EH Davis Memorial Lecture, Australian Geomechanics Society.

Indraratna, B., Balasubramaniam, A. S., and Balachandran, S. (1992). Performance of test embankment constructed to failure on soft marine clay. J. Geotech. Eng., ASCE, Issue 118, 1992, pp12-33.

Indraratna, B. and Chu, J (Editors). (2005). Ground Improvement - Case Histories, Elsevier.

Lambe, T.W. and Whitman, R.V. (1969). Soil Mechanics, Wiley, New York.

Mesri, G. (1991). Prediction and performance of earth structures on soft clay, Proc. First Intl Conf. on Coastal Development, Volume 2, Yokohama Japan, ppG2.1-G2.16.

Mesri, G. (1975). New design procedure for stability of soft clays, Discussion, J. of the Geotech. Eng. Div., ASCE, Volume 101, Issue 4, pp409-412.

Mesri, G., Lo, D.O.K., and Feng, T.W. (1994). Settlement of embankment on soft clay. Keynote Lecture, Settlement 94, Geotch. Spec. Publ. Issue 40, 1, Texas A and M University, College Station, Tex., 1, pp8-56.

Nakase, A. and Takemura, J. (1989). Prediction of trial embankment built to failure. Proc. Trial Embankments on Malaysian Marine Clays, Volume 2, The Malaysian Highway Authority.

Poulos, H.G., Lee, C.Y., and Small, J.C. (1989). Predicted behaviour of a test embankment on a Malaysian marine clay. Proc. Trial Embankments on Malaysian Marine Clays, Volume 2, The Malaysian Highway Authority.

Robertson, N.F. (1984). The use of monitoring for roadworks construction control. Proc. 4th Australia - New Zealand Conference on Geomechanics, Perth, pp 321-328.

Terzaghi, K. and Peck, R.B. (1967). Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley & Sons.

Wijeyakulasuriya, V., Hobbs, G. and Brandon, A. (1999). Some experiences with performance monitoring of embankments on soft clays. Proc.8th Australia New Zealand Conference on Geomechanics, Hobart, pp 2-783 – 2-788.

Wong (2006). Preload design, Part 1 – Review of soil compressibility behaviour in relation to the design of preloads. Australian Geomechanics Society Sydney Chapter 2006 Symposium, pp27-32.

Wong, P.K. (2007). Preload design to reduce post-construction creep settlement. Soft Soils and Ground Improvement Workshop, Proc. 10th Australia New Zealand Conference on Geomechanics, Brisbane.

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion

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Golder Associates, Sydney, NSW, Australia

Abstract:-

The Port Botany Expansion (PBE) project involves the construction of an extension to the existing port in Sydney, Australia. The transition between the new structures and the existing Brotherson Dock (EBD) structures is a critical aspect of the geotechnical design. The Client, Sydney Ports Corporation (SPC), specified tight differential movement and settlement limitations for the transition between the new and old structures, including a stringent 5mm differential movement limit (horizontal and vertical) up to 20 years after handover of the new terminal. The subsequent geotechnical and structural design of transition structures included measures to comply with these movement limits. The Main Contractor, Baulderstone Hornibrook – Jan de Nul (BHJDN) are carrying out construction trials, in-situ testing and movement monitoring to assess performance against Golder Associates' (Golder) design predictions. This paper describes the key design issues, design approach and verification processes established to confirm the predicted behaviour of the structures and surface infrastructure in order to satisfy criteria extending up to 50 years following handover.

Introduction

The PBE project comprises a new container terminal on the north-eastern shore of Botany Bay, about 12 kilometres south of the Sydney CBD. The new terminal lies between the existing port and the parallel runway at Sydney International Airport, extending approximately 550 metres west and 1,300 metres north of the northern quay of the EBD container terminal and covering an area of approximately 63 hectares. The project includes reclamation of the terminal area from Botany Bay and construction of 2 kilometres of berth structures, breakwaters, bridges, access corridors and revetments associated with the port facility.

In this paper the writer discusses the geotechnical design of the transition structures that connect the new and existing container terminals. Two anchored caisson structures weighing 2100t (Main Blockwork) and 684t (Infill Blockwork) form the transition with the EBD. Geotechnical analyses and design work included all aspects of geotechnical stability and serviceability design of retaining structures, including assessing the effect on the existing structures.

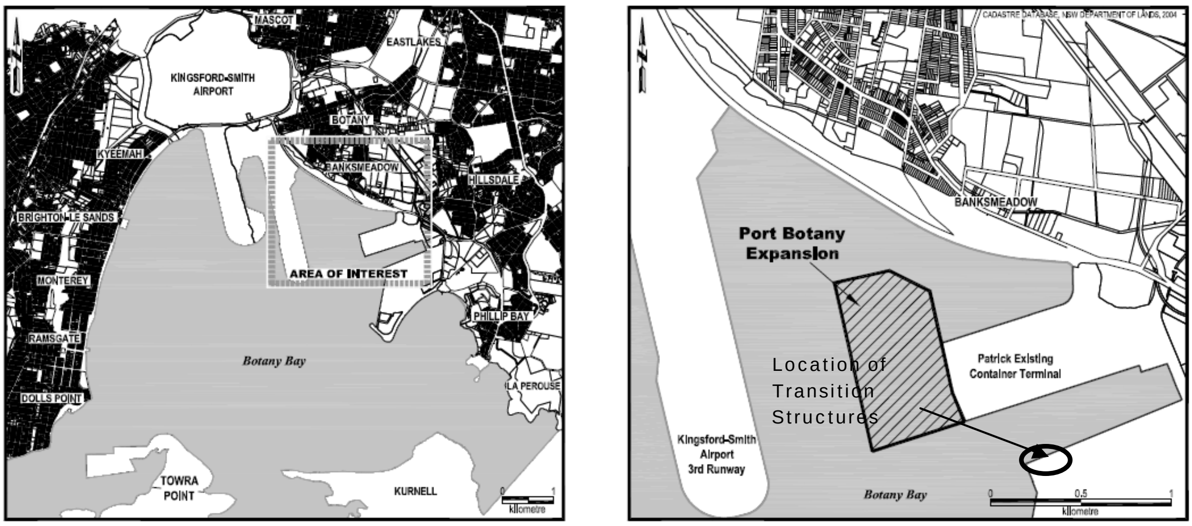

Figure 1: Plan View of Port Botany Expansion Project

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

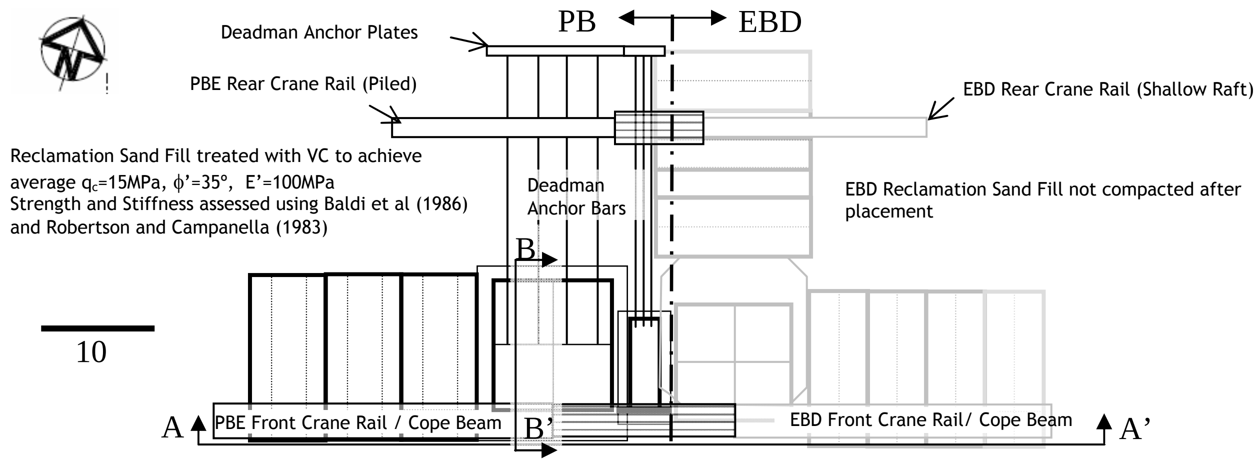

Figure 2: Plan View of Transition Structures

The Golder design team used a wide range of geotechnical software packages, including PLAXIS, Slope/W and Settle 3D. Other issues considered during the design included the effect of vibrocompaction close to the structures and assessment of bearing capacity under seismic loading. An important aspect of the design involved modelling the performance of the existing berth structure; designed to different design standards than the new structure as reported by Moss-Morris (1981). There was uncertainty regarding the historic loading regime applied to the existing structure. This lead to discussion on how the previous loading of the structure could affect ongoing movements once the new port terminal is in operation.

Geological model

The design team reviewed the geology of Botany Bay as part of the overall design, the geology of the locality is well summarised by Thorne (1985). As part of the design, a geotechnical model for the transition structures was developed as shown in section A-A' below:

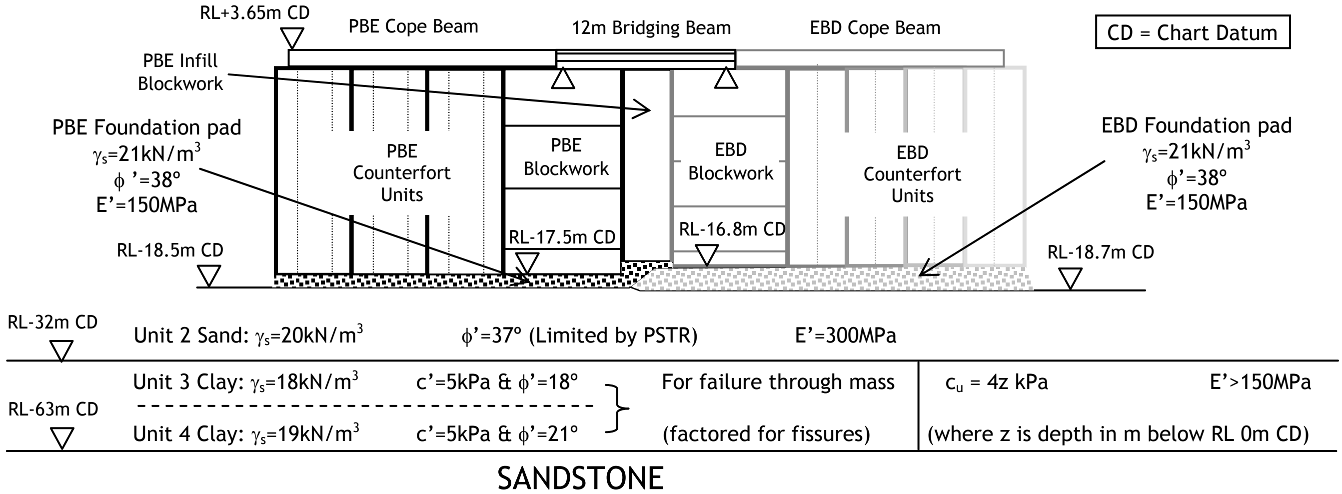

Figure 3: Section A-A': Geotechnical Model and Geotechnical Design Parameters for Transition Structures

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

At the location of the transition structures approximately 15m of dense Unit 2 sands exist over the Unit 3 fissured clays. This profile meant that fissured clays had less impact on the design of the transition structures than some counterfort walls on the site. The design of the counterfort structures incorporates a sand foundation trench, compacted using vibrocompaction. Discussion of the influence of fissured clays on the design of structures has been discussed extensively by Thorne (1984). Further discussion of the fissured clays in relation to the recent PBE design work would be valuable to add to the existing knowledge of these materials.

Design Requirements

The design of the transition structures needed to satisfy stability, settlement and movement criteria, which SPC specified in the Project Scope and Technical Requirements (PSTR). This document also specified the surface, crane and mooring loads that the new berth structures need to carry, including point loads to be applied to the EBD blockwork structure. Discussions were held between Golder, the structural designers Hyder Consulting and Scott Wilson and SPC to develop appropriate load cases for geotechnical analysis of the EBD blockwork structure, taking into account structural redistribution of loads.

2D PLAXIS modelling included different loads before and after construction of the PBE transition blockwork structures, taking into account structural re-distribution effects. The design assumed that the use of the western end of the EBD will not change significantly after construction of the new terminal. Additional live loading of the existing blockwork will be primarily due to crane rail load transferred across the bridging beam between new and old terminals. Operational loads used for the initial design development are shown in Table 1:

Table 1: Loads for PBE Transition Blockwork Structures and EBD Blockwork Structures

<table>
<tr><th>Proposed PBE Transition Blockwork Structures</th><th colspan="2">Sustained Loads</th><th colspan="2">Transient Loads</th></tr>
<tr><td>Surface Loading between the front and rear crane rails</td><td colspan="2">40kPa</td><td colspan="2">40kPa</td></tr>
<tr><td>Surface Loading landward of the rear crane rail</td><td colspan="2">60kPa</td><td colspan="2">60kPa</td></tr>
<tr><td>Vertical Crane Load on the front crane rail</td><td colspan="2">350kN/m</td><td colspan="2">970kN/m</td></tr>
<tr><td>Horizontal Crane Load on the front crane rail</td><td colspan="2">N/A</td><td colspan="2">97kN/m</td></tr>
<tr><td>Vertical Crane Load on the rear crane rail</td><td colspan="2">460kN/m</td><td colspan="2">Up to 1000kN/m</td></tr>
<tr><td>Horizontal Crane Load on the rear crane rail</td><td colspan="2">N/A</td><td colspan="2">Up to 100kN/m</td></tr>
<tr><td>Horizontal Mooring Load on the cope beam</td><td colspan="2">N/A</td><td colspan="2">91kN/m</td></tr>
<tr><th>Existing EBD Blockwork Structure</th><th colspan="2">Sustained Loads</th><th colspan="2">Transient Loads</th></tr>
<tr><td>Surface Loading landward of the front crane rail</td><td colspan="2">40kPa</td><td colspan="2">40kPa</td></tr>
<tr><td>Vertical Crane Load on the front crane rail</td><td>290kN/m [1]</td><td>385kN/m [2]</td><td>N/A</td><td>507kN/m [2]</td></tr>
<tr><td>Horizontal Crane Load on the front crane rail</td><td>29kN/m [1]</td><td>39kN/m [2]</td><td>N/A</td><td>51kN/m [2]</td></tr>
<tr><td>Vertical Crane Load on the rear crane rail</td><td>125kN/m [1]</td><td>250kN/m [2]</td><td>N/A</td><td>250kN/m [2]</td></tr>
<tr><td>Horizontal Crane Load on the rear crane rail</td><td>N/A</td><td>N/A</td><td>N/A</td><td>26.5kN/m [2]</td></tr>
<tr><td>Horizontal Mooring Load on the cope beam</td><td>25kN/m [1]</td><td>25kN/m [2]</td><td>25kN/m [1]</td><td>25kN/m [2]</td></tr>
</table>

1. ‘Historical Loads’ used to assess performance of EBD Blockwork prior to the construction of PBE.
2. ‘Future Loads’ used to assess performance of EBD Blockwork after handover of PBE (operational use of the new port).

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

The sustained loads shown above are for "normal" crane operations. The transient loads assume that a 1 in 500 year storm causes higher loads as it pushes cranes and moored vessels away from the wharf structures. The most onerous load case used for the design of the transition blockworks included both transient crane loads and transient mooring loads.

The PSTR set the following movement limits for the PBE blockwork structure:

- Total vertical settlement of PBE blockwork to be less than 40mm after 20 years.
- Total horizontal movement of PBE blockwork to be less than 40mm after 20 years.
- Vertical differential settlement between EBD and PBE blockworks to be less than 5mm over 5m after 20 years.
- Horizontal differential settlement between EBD and PBE blockworks to be less than 5mm over 5m after 20 years.

In addition, the PSTR specified the stringent differential movement and settlement criteria to provide crane beam continuity between the new and old docks. If future movement exceeds these limits then the crane rails would need to be reset, resulting in potential disruption to the new and old terminal operators and potential commercial ramifications.

The PSTR required that the design had to achieve the following minimum Factors of Safety (FOS):

- Bearing capacity: 3.00
- Sliding and Overturning: 2.00
- Global Stability: 1.50 Circular / 1.40 Non-circular / 1.10 Design Earthquake Event (1 in 1,000AEP Earthquake)

Design Solution

PLAXIS software was used to model the movement of the structure and earth pressures acting on the structure at different times during construction and the operational life of the structure. Global stability of the structures was assessed using SLOPE/W software and spreadsheets were used to check the stability of the structure using limit equilibrium analyses for sliding, overturning and bearing capacity.

Deformation Analyses

The design team used PLAXIS software to assess the total and differential deformation behaviour of the EBD and PBE blockwork structures and to estimate the earth pressures acting behind the structures; both during construction and in operation. At an early stage of the design development the design team identified that meeting the differential movement limits between the existing and new structures would be the critical aspect of the design of the transition blockwork. In the 2-D plane-strain analysis a strain hardening model was used for granular materials to limit heave and soft soil creep models for Unit 3 and 4 clays.

The design team back analysed the performance of the EBD counterfort structures using PLAXIS and Settle3D to select the most suitable deformation parameters for the design of the PBE structures. The effect of different deformation parameters for the soil units was modelled, based on laboratory testing results, the design of the EBD as discussed in technical papers historic design reports and statistical assessment. The design deformation parameters were then calibrated to match the measured movement of the EBD counterforts at four locations along the existing dock using the original design loading.

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

The movement response of the EBD blockwork due to the new load regime was assessed, based on the design deformation parameters and the design loadings. With the predicted movement of the EBD blockwork available the deformation behaviour of a free-standing PBE blockwork structure was modelled and it was found that horizontal movements of the new blockwork would not comply with the horizontal differential requirements. To reduce movement of the PBE blockwork lightly pre-stressed deadman anchors were added to the proposed structure, in a movement reducing role. The stiffness of these anchors had to be carefully assessed. If their response was too rigid, it could lead to unacceptable differential movements and high structural forces in the PBE blockwork unit and front cope beam. The design has since been completed using four 50mm diameter Maccolloy post-tensioning bars. As the design proceeded the geotechnical team worked collaboratively with the structural designers to ensure compatibility between the movement response and load inputs to the structural and geotechnical models, including the anchorage system.

Even with the deadman anchors, the structures were close to the PSTR compliance limits for differential movement. The geotechnical and structural design team resolved this by reviewing two aspects of the design:

1. Review of the input load cases and the input load distribution assumptions.
2. Extending the length of bridging beam between the new and old structures from 8.6m to 12m.

The load case review found that the transient loadcase used for the PLAXIS modelling included a hypothetical combination of loads; a transient load caused by a crane operating in high winds could not occur at the same time as the transient mooring load under normal port operating procedures. This resulted in a reduction of the maximum horizontal transient loading (Horizontal Crane Load + Horizontal Mooring Load) from 191kN/m to 128kN/m, which caused a similar proportional reduction in predicted horizontal movement of the new blockworks. Extension of the bridging beam helped to improve the differential performance, to achieve the PSTR differential limit criteria.

Assessment of conventional limit equilibrium seismic bearing capacity was supplemented by a displacement based seismic analysis. Seismic displacement criteria were subsequently adopted as the main performance criteria. The dynamic displacement of the PBE blockwork structure was assessed under the design earthquake event (PGA=0.14g) using a 2D dynamic seismic PLAXIS analysis. Interestingly, this analysis showed a similar movement mechanism to recorded movements of port caisson units after the approximately 0.51g Kobe Earthquake (Soga, 1998). The PLAXIS model predicted minimal settlement and a seaward translation of the structure of approximately 30mm. The analysis also showed a reduction of earth pressures towards K_a, during the earthquake which supported the use of a modified Mononobe-Okabe incremental seismic force for sliding and overturning checks (Value used = 130%ΔP_{ae}).

A summary of the static movement predictions from PLAXIS for the PBE blockwork structure is shown in Figures 4 and 5:

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

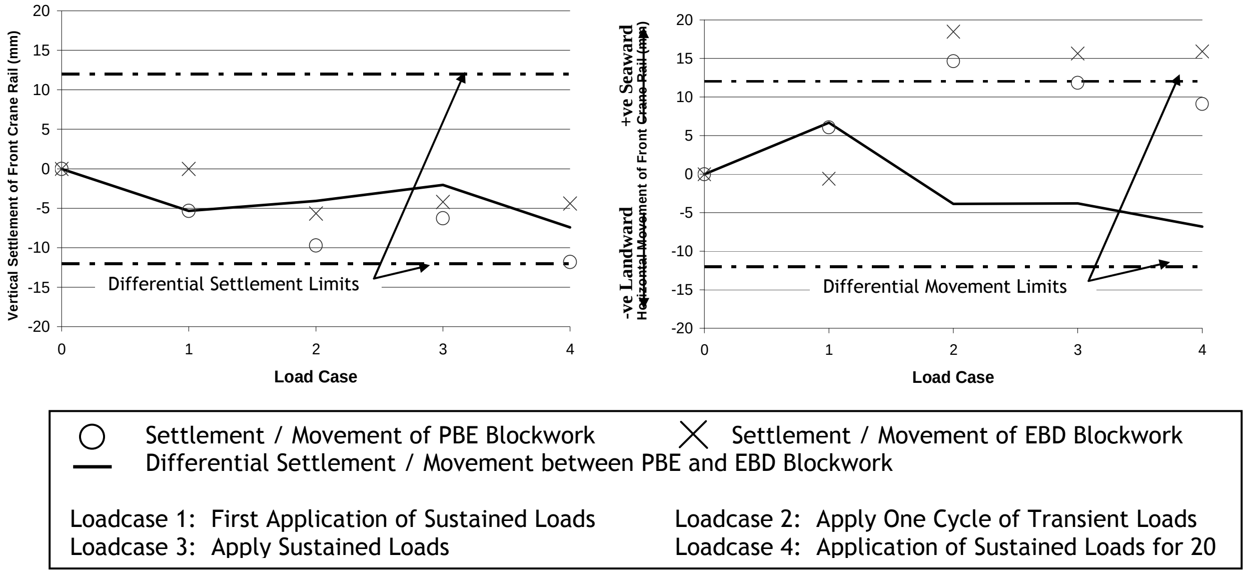

Figure 4: Predicted Movement of New and Existing Structures AT Section B-B' (Figure 3)

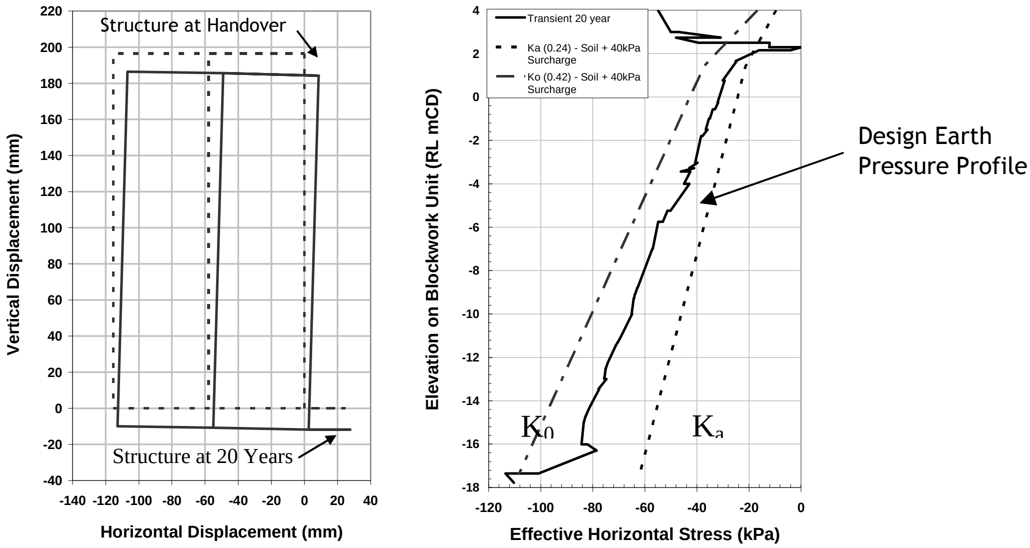

Figure 5: Earth Pressures and Block Deformation: 20 Year Consolidation after Application of Transient Loads

Stability assessment

Spreadsheet calculations were developed to assess the stability of the structures for sliding, overturning and bearing capacity mechanisms at the base of the structure and at the base of the gravel pad. The earth pressures used in the spreadsheets were matched with the earth pressure envelope predicted by the PLAXIS model. The best match occurred using a K_0 earth pressure coefficient in combination with a wall interface friction coefficient of 0.5.

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

Base roughening of the blockwork units was also analysed to assess FOS against sliding failure of the blocks. A system of shear teeth on the underside of the base slab was developed to enable a base interface friction coefficient of 1.0 to be used. The design achieved PSTR compliant FOS values and meets the requirements of AS 4678 (2002). Although this Standard adopts a limit state methodology, it supports alternative design approaches including use of global “lumped” geotechnical resistance factors.

Temporary works

Temporary support was required for the EBD counterfort structures to the north of the EBD Blockwork as these were founded at approximately RL-13.5mCD, compared to the required excavation for the PBE blockwork of RL-18.5mCD, a retained height of up to 5m. PLAXIS and hand calculations were used to assess the deformation and shear and moment capacity of a cantilever soldier pile wall, using 10No 840mm diameter, 20mm thick tubular steel piles at 1.5m centres. Movement trigger levels were developed to monitor the performance of the wall as described in Section 4.5.

Effect of Vibrocompaction on Earth Pressures

The potential impact of vibrocompaction (VC) adjacent to retaining structures was reviewed as part of the overall PBE design. It was recognised that the earth pressures acting on structures change with time and are affected by the method of placement of reclamation fill, compaction type and energy and structural movements. A review of published literature found no published method of assessing the impact of VC on retaining structures. The geotechnical team produced a design assessment of the impact of VC, based on previous work by Greenwood et al (1984) and Massarsch and Fellenius (2002). This design assessment indicated that structures are unlikely to experience stressing beyond operational earth-pressures due to VC improvement from q_c 5MPa to 15MPa when carried out beyond a distance of approximately 4 to 5m from the back of the structure.

During the design process the importance of site trials was recognised to assess suitable compaction methods immediately behind the wall. In addition, the potential for use of reduced compaction criteria immediately behind the wall was explored. The objective of this was to balance the risk of creating unacceptably high earth pressures against achieving the required backfill strength and stiffness and thereby acceptable movement performance of the structure.

The risk of locking in higher than designed for stresses was mitigated by the following strategy:

- Consideration of alternative types/sizes of compaction equipment;
- Assessment of an amended compaction procedure, including changing offset distance or lift rate;
- Investigating the possibility of revised compaction criteria immediately behind structures;
- Verification of assumptions at early stage of production works, including CPTs, Pressure Cell monitoring and introduction of Hold Point before commencing production compaction; and
- Pressure cell locations were moved to the eastern end of the East-West berth to allow the effect of production compaction to be verified at an early stage of the construction.

Site trials subsequently proved that reduced energy VC compaction produced earth pressures and vibrations that were acceptable for VC probes located to within 2.5 m of moveable structures. The author plans to document the effects of VC and extensive trial data from the Port Botany site as part of a separate, more detailed technical paper.

Case History

Geotechnical Design of Transition Structures for the Port Botany Expansion (continued)

Verification Testing

To provide confidence that the performance of the as-constructed structures is similar to the design predictions, monitoring of the structures is being carried out as an integral part of the construction process. This monitoring considers both the movement of the structure (using tiltmeters, inclinometers and surface monitoring points) and the earth pressures acting behind the structure (using five earth pressure cells down the rear face of the structure). For different construction stages the movement of the structure and the earth pressures acting on the structure were assessed, as shown in Figure 5. Trigger levels were developed for movement and earth pressures using a “traffic light system” to help communicate the action required during construction if movements and/or earth pressures approach or become higher than the design values. This is especially important as there are over 1,000 monitoring points on the PBE site and without an effective action plan to respond to the data collected, critical information could get missed. Currently movements are in accordance with the design predictions. It is anticipated that more information will be provided in a separate paper at a later date.

Acknowledgments

The writer wishes to thank BHJDN, Hyder and SPC for permission to publish data produced during the PBE design. The author also wishes to thank Philip Davies and Craig Curnow of Golder Associates for comments and support on this paper.

References

Moss-Morris, A. and Hodge, N.M. (1981) Design and Construction of Quay Walls for a Container Terminal Founded on Fissured Clays, *25th International Conference of Navigation*, Edinburgh. Section 11, Vol 4 pp 557-568.

Thorne, C.P. (1985) Deep marine fissured clays, *Engineering Geology of the Sydney Region*, Edited by PJN Pells, Balkema, pp 63-79.

Thorne, C.P. (1984) Strength assessment and stability analyses for fissured clays, Geotechnique, Vol 34, No. 3, pp 305-322.

Baldi, G., Belotti, R., Ghionna, V., Jamiolkowski, M., and Pasqualini, E. (1986) Interpretation of CPT’s and CPTU’s, 2nd part: Drained Penetration of Sands, *Fourth International Geotechnical Seminar*, Singapore, pp 143-156.

Robertson and Campanella (1983) Interpretation of Cone Penetration Tests (Part I), Canadian Geotechnical Journal, Vol. 20, No. 4.

Greenwood, D.A. and Kirsch, K. (1984). Specialist ground treatment by vibratory and dynamic methods. *Piling and Ground Treatment*, Thomas Telford Ltd, London, pp 17-45.

Massarsch, K.R. and Fellenius, B.H. (2002). Vibratory compaction of coarse-grained soils, Canadian Geotechnical Journal, Vol. 39, pp 695-709.

Soga, K. (1998). Soil liquefaction effects observed in the Kobe earthquake of 1995, Geotechnical Engineering, Vol. 131, Jan, pp 34-51

NEWS

17th Southeast Asian Geotechnical Conference in Taipei

17th Southeast Asian Geotechnical Conference (17SEAGC) was held in Taipei from May 10th to 13th under the auspices of Southeast Asian Geotechnical Society (SEAGS) and Taiwan Geotechnical Society. The main themes of the conference were “Geo-engineering for Natural Hazard” and “Mitigation and Sustainable Development.” The conference venue was a beautiful international convention center. This series of conference is the most important event of the Southeast Asian Geotechnical Society.

This society was originally founded in 1967 by Dr. Za-Chieh Moh as a regional society to cover Thailand, Malaysia, Singapore, Philippines, Hong Kong and Taiwan. It has now a membership of over 260. Its members are very active in soil mechanics and foundation engineering, engineering geology, rock mechanics, geoenvironmental engineering, and geosynthetic engineering. SEAGS has been making remarkable contributions to the international geotechnical community by, for example, hosting Asian Regional Conference on Soil Mechanics and Geotechnical Engineering two times in the past (Bangkok and Singapore), sponsoring technical committees (original TC39 on Geotechnical Engineering for Coastal Disaster Mitigation and Rehabilitation, now TC303), and publishing Geotechnical Engineering Journal.

17SEAGS collected more than 130 papers and 300 participants not only from the Southeast Asian Region but also from North America, Europe, Africa, Australia and many other Asian Countries, e.g., Japan, Korea, China, Iran, India, Kuwait and Kazakhstan, together with our international President Briaud from USA.

Photograph below shows the delegates appearing in the opening ceremony of the Conference.

NEWS

5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor I.M. Idriss

This conference took place from May 26-29, 2010, in San Diego, California in USA. As shown in its title, this conference is the fifth occasion in its history that has been organized since 1981 by Prof. Shamsher Prakash of Missouri University of Science and Technology. One of the aims of this occasion in this year was to respect Prof. I.M. Idriss who made long and tremendous contributions to the development and practice of geotechnical earthquake engineering. The conference attracted 366 paper submissions and 417 participants from 51 countries. Presentations and discussions started at 8 A.M. and lasted till 9 P.M.. Participants were able to fully enjoy this good opportunity to exchange information and opinions in the field of geotechnical earthquake engineering and disaster mitigation. During the conference, the Shamsher Prakash Award of 2010 for young professionals was presented to Tara Hutchinson from USA and Jean-Francois Semblat from France for their research achievements, while Allen William Cadden from USA and Zygmunt Lubkowski from UK were honored for their excellent practices. A post-conference tour to the University of California San Diego Englekirk Center was enjoyed by 90 of the conference participants. The tour was a collaboration between the University of California San Diego, University of California Santa Barbara, University of Texas, and University of California Los Angeles. Participants observed demonstrations of some of the most impressive and significant large-scale earthquake engineering research equipment in the U.S.

Photographs below present the conference chairman, Prof. Prakash, together with his assistants Tammy Mace and Lindsay Bagnall, and participants gathering before a session of the ongoing discussion.

NEWS

International Conference on Geosynthetics

General information

The 9th International Conference on Geosynthetics (9th ICG) was held in Guarujá, Brazil, from 23rd to 27th of May in 2010. The city of Guarujá is located on the island of Santos Amaro, and is known as a beautiful seaside resort called the Pearl of the Atlantic. The 9th ICG was held in a convention center in the Sofitel Jequitimar Hotel, which is one of the best convention centers on the coast line of Brazil.

As the primary international geosynthetics event, this series of conference has been organized by IGS once every four years. The 9th ICG took place for the first time in the southern hemisphere, and its chief organizers were the Brazilian Association of Geosynthetics (IGS Brazil) and the Brazilian Association of Soil Mechanics and Geotechnical Engineering (ABMS). This conference was also supported by the International Geosynthetics Society (IGS), the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), and the Brazilian Association of Nonwoven and Technical Textiles (ABINT).

Number of participants

The conference attracted 814 geotechnical engineering academics and practitioners from 50 countries. The largest number of participants (250) was from Brazil, the host country. Asia was also well represented with 265 participants from 11 countries, among which 198 were from Taiwan.

Country	Participants	Country	Participants	Country	Participants	Country	Participants
ARGENTINA	19	FINLAND	2	LITHUANIA	5	SINGAPORE	3
AUSTRALIA	14	FRANCE	22	LUXEMBOURG	1	SLOVAKIA	1
AUSTRIA	3	GERMANY	25	MALAYSIA	6	SOUTH AFRICA	8
BELGIUM	6	GHANA	1	MEXICO	10	SPAIN	6
BRAZIL	304	GREECE	1	NETHERLANDS	2	SWITZERLAND	2
CANADA	22	HONG KONG	1	NEW ZEALAND	8	TAIWAN	13
CHILE	14	INDIA	8	NORWAY	9	THAILAND	3
CHINA	6	INDONESIA	1	PANAMA	2	TURKEY	4
COLOMBIA	6	IRAN	4	PARAGUAY	1	UKRAINE	3
COSTA RICA	5	IRELAND	2	PERU	19	UNITED KINGDOM	18
CROATIA	6	ISRAEL	4	PORTUGAL	8	UNITED STATES	69
DENMARK	2	ITALY	17	QATAR	1	URUGUAY	1
ECUADOR	3	JAPAN	37	ROMANIA	5	VENEZUELA	2
EL SALVADOR	1	KOREA	7	RUSSIAN FED.	3	Total	756

Accompanying persons	33
Total participants	789

NEWS

International Conference on Geosynthetics (Continued)

<u>Technical program</u>

The conference organizing committee received 317 technical papers from 42 countries, while 200 papers were selected for the oral presentation and 117 papers were invited to participate in poster sessions. Technical sessions were divided into 16 different categories so as to reflect the wide varieties of Geosynthetics usage in the practice.

Discussion sessions on 12 themes were also organized. Discussion session began with 20-minutes presentations by the Theme Speakers, followed by discussions and questions from the audience.

Several training lectures also made part of the technical program of the 9ICG, which characterized the special features of ICG as compared with other international conferences. The aim of the training lecture was to allow for attendees to get in contact with most of the specialists in specific topics and have the opportunity to learn and get some information about definitions and concepts, design methodologies, relevant aspects, and installation procedures among others regarding specific themes.

A technical exhibition was held with the participation of 60 companies to display development of product, equipments, software and methods of construction of geosynthetics.

List of Technical sessions

Best Journal paper session	Hydraulic applications of Geosynthetics
Drainage and Filtration	Innovative Products and Applications of Geosynthetics
Durability	Meeting the Industry Session
Discussion session (divided into 12 sessions)	Mining
Environmental Application	Other Application of Geosynthetic Reinforcement
Embankments on Soft Soils	Geosynthetics in Highway and Railways
Geosynthetics Testing and Properties	Piled Embankments
Soil Geosynthetic Interaction	Retaining Walls and Steep Slopes

* Geotextiles & Geosynthetics, **Geosynthetics International

Keynote lecture by Dr. D. Cazzuffi (Italy)

Technical exhibition

NEWS

International Conference on Geosynthetics (Continued)

Special lectures

Seven special lectures were given during the conference. Just after the welcome reception, two important lectures were delivered. The welcome lecture on case history of Brazilian research and practice on Geosynthetics was presented by Dr. Sandro S. Sandroni of Brazil and then Dr. Giroud of US gave a Prestigious Lecture on criteria for geotextile and granular filters. The Giroud Lecture on the practical application of geosynthetics for the mitigation of the natural disasters by Prof. Brandl (Australia) opened the first day of the conference. Dr. Daniele Cazzuffi opened the second day of the conference as one of the Keynote Lectures about the Geosynthetics barrier system for dams. In the second day, Prestigious Lecture on reinforced soil retaining walls by Robert D. Holtz was also presented. Two more keynote lectures were also delivered at the beginning of third and fourth days, respectively. Prof. Steve W. Perkins presented the keynote lecture on the applications of geosynthetics to the pavement. Prof. Andy Fourie delivered the last keynote lecture of the conference at the beginning of fourth day on the geosynthetics application for the improvement of performance of mining infrastructures.

Social events

Apart from the scientific program, attractive social events were also prepared. Participant enjoyed Brazilian happy- hour with the opportunity to interact with and visit the booths of the exhibitors with appetizers and music performance at the end of first day. Some of the participants played a traditional football match (beach-soccer) on the second night of the conference. The participants had the opportunity to socialize, dance, and taste some of the plentiful and delicious Brazilian cuisine during the conference dinner at the Guarujá Late Clube.

IGS council and next ICG

During the conference in Guarujá, new council term members were elected to serve for the 2010-2014 term. The elected officers are: Dr. Jorge G. Zornberg of USA as the President, Dr. Russell Jones of UK as the Vice President, and Prof. Fumio Tatsuoka of Japan as the Immediate Past President. Berlin of Germany was decided as the venue of the 10th ICG to be held in 2014.

NEWS

GeoMos2010 Conference in Moscow

An International Conference "Geotechnical Challenges in Megacities" (GeoMos2010) was held in Moscow, Russia, from 7 to 10 of June, 2010, under the auspices of ISSMGE and its Russian daughter-society with co-sponsorship of five Technical Committees of ISSMGE.

The organizers and general sponsors of GeoMos2010 were the Gersevanov Research Institute of Bases and Underground Structures (NIIOSP), Moscow, and NPO "Georeconstruction-Fundamentproject" (GRF), Saint-Petersburg. Technische Universität Darmstadt was another general sponsor. There were eight other sponsors.

The Conference was attended by ISSMGE President Professor Jean-Louis Briaud, Secretary General Professor Neil Taylor, Past Presidents Professors P. Pinto and M. Jamiolkowski together with many other prominent professionals. The total number of participants was 264.

One of the aims of GeoMos2010 was to strengthen the ties between local and international professionals; that is why the Conference was declared bi-lingual (English and Russian). The simultaneous translation was provided for all the sessions of GeoMos2010, and the papers in the Conference Proceedings were published in either language at the author's will.

GeoMos2010 was preceded by the first meeting of the newly elected ISSMGE Board and the sponsoring TC meetings on June, 6th.

During the conference 14 lectures were delivered by famous researchers: V. Petrukhin and M. Jamiolkowski (Keynote lectures); J.-L. Briaud, H. Brandl, R. Katzenbach, R. Frank, H. Schweiger, R. Kastner, G. Viggiani, V. Ulitsky, P. Pinto, I. Vaníček, K. Pitilakis and A. Negro (Invited lectures). For oral presentation in two parallel sessions 67 papers were selected also. Published papers were reviewed in 6 general reports (each of them presented by two co-reporters working with papers in Russian and English correspondingly).

The conference scope embraced multi-directional and multi-disciplinary themes: "Construction in restrained urban areas" (topics – "Foundations for high-rise buildings", "Deep excavations, retaining structures, diaphragm walls", 'Tunnels for underground transport infrastructures and networks" and others), "Preservation of existing structures & soil-structure interaction" (topics – "Effect of new buildings and constructions on underground structures", "Effect of new underground structures on existing buildings and networks", "Preservation of historical buildings" and others) and "Urban environmental geotechnics" (topics – "Geofailures & risk assessment", "Geological risks in urban planning", "Construction on contaminated soils", "Geotechnical sustainability" and others). A number of social events were held to provide opportunities for GeoMos2010 participants to communicate, relax and get good impression of old and new Moscow.

Three days of sittings were concluded by 2 parallel technical visits, the first to Moscow-City Business-Center, which is the site of many new high-rise buildings, and the second to the site of underground construction at "Sokol" station of Moscow Metro.

The conference was accompanied by a technical exhibition that showed 17 companies from 4 countries.

The Conference Proceedings include 5 volumes, the first being devoted to lectures (12 in English, 2 in Russian), the next two including 119 papers in English and the rest two including 125 papers in Russian. The authors represented 51 countries of all the inhabited continents; the total amount of pages exceeds 2000.

The work of the conference was highlighted by Russian technical journals, magazines and newspapers.

NEWS

GeoMos2010 Conference in Moscow (Continued)

Chairman of the Organizing Committee Professor Petrukhin, NIIOSP director, and ISSMGE Past Presidents Professor Pinto at GeoMos2010 opening

ISSMGE Vice-President for Europe
Professor Ivan Vaníček at Technical visit

ISSMGE President Professor Jean-Louis Briaud
aided by interpreter sharing his impressions
of GeoMos2010 in two languages

NEWS

2010 Shamsher Prakash Award

2010 Shamsher Prakash Research Award has been won by David Masin of Charles University in Prague, Czech Republic. He has been cited for his research on soil behavior, development and evaluation of constitutive models and for their application for solving boundary value problems in geotechnical engineering. For nominations for 2011 Research Award, visit the website: www.yoga10.org.

NEWS

Guidelines for Seismic Downhole Testing

ISSMGE Technical Committee No 10: Geophysical Testing in Geotechnical Engineering (1996-2004) had as part of its brief, the task of drafting guidelines for geophysical techniques where no other national or international standards or codes of practice exist.

TC 10 saw a need for an independent set of guidelines for seismic downhole testing, to be used in specifying work, checking testing procedure, data evaluation and quality assurance. There were, at that time, few standards for geophysics measurements and most procedures in use were evolved from research work (some commercial some academic) that developed the procedures.

This document was expected to be the first of these guidelines and concerns the use of the Seismic Cone to measure downhole seismic wave propagation velocities. It was originally started by R.G. Campanella, continued by A.P. Butcher with input from R.G. Campanella, A.M. Kaynia, and K.R. Massarsch, and other members of TC 10.

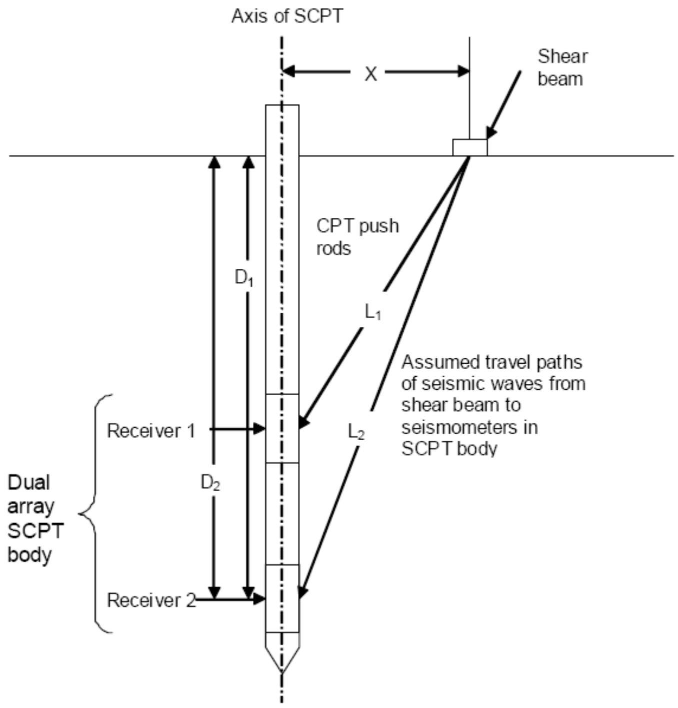

Figure 1: schematic diagram of the dual array seismic cone test with required dimensions, D_1, D_2, and X

The guideline was prepared as a supplement to the International Reference Test Procedure (IRTP) for the electric Cone Penetration Test (CPT) and the Cone Penetration Test with Pore pressure (CPTU) as produced by the ISSMGE TC16. The document therefore follows, and should be used with, the CPT IRTP (1999) but includes comments and recommendations with additional information and enhancements that can improve the quality of data and/or aid interpretation of the data. Several aspects of the guidelines can also be applied to other seismic tests, such as the seismic cross-hole test.

Reference: Butcher et al. (2005). Seismic cone downhole procedure to measure shear wave velocity. Guideline prepared by ISSMGE TC10: Geophysical Testing in Geotechnical Engineering. International Society for Soil Mechanics and Geotechnical Engineering. 5 p.

The guidelines are accessible from the ISSMGE web site: (URL link to be decided soon).

Event Diary

ISSMGE SPONSORED EVENTS

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

2010

7th International Conference on Physical Modelling in Geotechnics ICPMG 2010
Date: 28 June - 1 July 2010
Location: ETH Zurich, Honggerberg Campus , Zurich, Switzerland
Language: English
Organizer: ETH Zurich
• Contact person: Laios Gabriela
• Address: ETH Zurich, Institute for Geotechnical Engineering
8093 Zurich
Switzerland
• Phone: 41 44 6332525
• Fax: 41 44 6331079
• E-mail: info@icpmg2010.ch
Website: www.icpmg2010.ch

International Symposium on Geomechanics and Geotechnics: From Micro to Macro
Date: 10 - 12 October 2010
Location: Tongji University , Shanghai, China
Language: English
Organizer: Tongji University
• Contact person: Prof. Mingjing Jiang
• Address: Dept. of Geotechnical Engineering, Tongji University
200092 Shanghai
China
• Phone: 86-21-65980238
• Fax: 86-21-65980238
• E-mail: mingjing.jiang@tongji.edu.cn
Website: geotec.tongji.edu.cn/is-shanghai2010/

Bangladesh Geotechnical Conference 2010; Natural Hazards and Countermeasures in Geotchnical Engineering (04-05 Nov)
Date: 4 - 5 November 2010
Location: Sheraton Hotel , Dhaka, Bangladesh
Language: English
Organizer: BSGE
• Contact person: Yasin, Sarwar J M
• Address: Professor, Civil Engg. Dept., BUET
1000 Dhaka
Bangladesh
• Phone: 01817036073 (cell)
• Fax: 880-2-9665639
• E-mail: bsge.hgs@gmail.com
Website: www.bsge-bd.org (under construction)

6th International Congress on Environmental Geotechnics
Date: 8 - 12 November 2010
Location: New Delhi, India
Language: English
Organizer: Indian Geotechnical Society
• Contact person: Dr. G. V. Ramana
• Address: Associate Professor, Department of Civil Engineering.
Indian Institute of Technology Delhi, Hauz Khas
110016 New Delhi
India
• Phone: 911126591214
• Fax: 911126581117
• E-mail: 6icegdelhi@gmail.com
Website: www.6iceg.org

Fifth International Conference on Scour and Erosion (ICSE-5)
Date: 8 - 10 November 2010
Language: English
Organizer: Geotechnical Institute of ASCE
• Contact person: Cathy Avila
• Address: 712 Bancroft Road, Suite 333
94598 Walnut Creek
California
United States of America
• Phone: 1-925-673-0549
• Fax: 1-925-673-0509
• E-mail: cavila@avilaassociates.com
Website: www.icse-5.org

International Symposium on Forensic Geotechnics of Vibratory and Natural Hazards
Date: 14 - 15 December 2010
Location: Indian Institute of Technology , Mumbai, Maharashtra, India
Language: English
Organizer: TC 302, IGS (India), IITB
• Contact person: Prof. G L Sivakumar Babu
• Address: Department of Civil Enginering
560012 Bangalore
Karnataka
India
• Phone: 00918022933124
• Fax: 00918023600404
• E-mail: gls@civil.iisc.ernet.in
Website:
civil.iisc.ernet.in/~gls/default_files/FGE_Full%20brochure.pdf

Event Diary (continued)

2011

5th International Conference on Geotechnical Earthquake Engineering (5-ICEGE)
Date: 10 - 13 January 2011
Location: Santiago de Chile, Chile
Language: English
Organizer: CGS, ISSMGE TC4
• Contact person: Secretariat 5ICEGE
• Address: Toledo Nº 1991, Postal Code 7500000
Providencia, Santiago
Chile
• Phone: 56-2-2746714
• Fax: 56-2-2742789
• E-mail: secretariat@5icege.cl
Website: www.5icege.cl/

7th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground
Date: 16 - 18 May 2011
Location: Roma, Italy
Language: English
Organizer: TC28 and AGI
• Contact person: Dr. Ing. Claudio Soccodato
• Address: Associazione Geotecnica Italiana, viale dell'Università 11
00185 Roma
RM
Italy
• Phone: 39064465569
• Fax: 390644361035
• E-mail: info@tc28-roma.org
Website: www.tc28-roma.org

The 3rd International Conference on Geotechnical Engineering for Disaster Mitigation and Rehabilitation 2011 (GEDMAR 2011) Combined with The 5th International Conference on Geotechnical and Highway Engineering
Date: 18 - 20 May 2011
Language: English
Organizer: JWG-DMR, Diponegoro University
• Contact person: Ir.H. Wuryanto MSc, Dr. Bagus Hario Setiadji
• Address: Indonesian Road Development Association (IRDA) of Central Java, Jl. Puri Anjasmoro Blok I.1 No 12
50144 Semarang
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• Fax: 62-24 7622785
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Website: reliability.geoengineer.org/GEDMAR2011/

XIV Asian Regional Conference on Soil Mechanics and Geotechnical Engineering
Date: 23 - 27 May 2011
Location: Hong Kong Poly University , Hong Kong, China, China
Language: English
Organizer: HKGES and CSE of HK Poly U
• Contact person: Miss Laurel Lau
• Address: Dept of Civil & Struc Eng, Hong Kong Polytechnic University, Hong Kong
Hong Kong
China
• Phone: 852 2766 6017
• Fax: 852 2334 6389
• E-mail: 14arc.2011@polyu.edu.hk
Website: www.cse.polyu.edu.hk/14arc

XV African Regional Conference on Soil Mechanics and Geotechnical Engineering - "Resources and Infrastructure Geotechnics in Africa: Putting theory into practice".
Date: 18 - 21 July 2011
Location: Maputo, Mozambique
Organizer: Soc. Moçambicana de Geotecnia
• Contact person: Prof. Carlos QUADROS, President of SMG, Dr Saturnino CHEMBEZE, Sec. Gen SMG
• Address: Mozambican Geotechnical Society, Av. 25 de Setembro nº 2526
Maputo
Mozambique
• Phone: 258 21322185
• Fax: 258 21322186
• E-mail: info@15arcsmge-maputo2011.com
Website: www.15arcsmge-maputo2011.com

Fifth International Symposium on Deformation Characteristics of Geomaterials (IS-Seoul 2011)
Date: 31 August - 3 September 2011
Location: Sheraton Grande Walkerhill , Seoul, Korea
Language: English
Organizer: ISSMGE(TC-29) and KGS
• Contact person: Prof. Dong-Soo Kim
• Address: Dept. of Civil & Environmental Eng., KAIST
305-701 Daejeon
Korea
• Phone: 82-42-350-5659
• Fax: 82-42-350-7200
• E-mail: is-seoul@kaist.ac.kr
Website: www.isseoul2011.org

XV European Conference on Soil Mechanics and Geotechnical Engineering "Geotechnics of Hard Soils - Weak Rocks"
Date: 12 - 15 September 2011
Location: Megaron Athens Int Conf Cntr , Athens, Greece
Language: English/French
Organizer: HSSMGE
• Contact person: Secretariat XV ECSMGE – Athens 2011
• Address: PO Box 26013
10022 Athens
Greece
• Phone: 30 210 6915926
• Fax: +30 210 6928137
• E-mail: athens2011ecsmge@hssmge.gr
Website: www. athens2011ecsmge.org

XIV Panamerican Conference on Soil Mechanics and Geotechnical Engineering (October) & V PanAmerican Conference on Learning and Teaching of Geotechnical Engineering, & 64th Canadian Geotechnical Conference

Event Diary (continued)

Date: 2 - 6 October 2011
Location: Sheraton Hotel Toronto, Ontario, Canada
Organizer: CGS

2012

11th Australia - New Zealand Conference on Geomechanics
Date: 15-18 July 2012
Location: Melbourne, Australia
(Please note that these dates still need to be confirmed.)

NON-ISSMGE SPONSORED EVENTS

2010

The 11th Congress of the International Association for Engineering Geology and the Environment. (IAEG2010)
Date: 5 - 10 September 2010
Language: English
Organizer: Clare Wilton
• Contact person: The Conference Company
• Address: PO Box 90 040
1142 Auckland
New Zealand
• Phone: 64 9 360 1240
• Fax: 64 9 360 1242
• E-mail: iaeg2010@tcc.co.nz
Website: www.iaeg2010.com

1st International Conference on Information Technology in Geo-Engineering (ICITG-Shanghai 2010)
Date: 16 - 17 September 2010
Location: Tongji University , Shanghai, China
• Contact person: Dr. Xiaojun Li
• Address: Secretary of ICITG-Shanghai 2010, Associate Professor,
School of Civil Engineering, Tongji University,
No.1239 Siping Road
Shanghai 200092
China
• Phone: Ph: 86-21-65985174
• Fax: 86-21-69585140
• E-mail: lixiaojun@tongji.edu.cn
Website: geotec.tongji.edu.cn/ICITG2010/default.html

XIII Colombian Geotechnical Congress and VII Colombian Geotechnical Seminar (
Date: 21 - 24 September 2010
Language: Spanish-English
Organizer: Colombian Geotechnical Society
• Contact person: JUAN MONTERO OLARTE
• Address: Calle 14 No 8-79 Of. 512 - Edificio Bolsa
11001000 Bogota D.C.
Colombia
• Phone: 57-1-3340270
• Fax: 57-1-3340270
• E-mail:
scg1@etb.net.co;scg1@colomsat.net.co;juanmontero17@etb.net.co
Website: www.scg.org.co

Workshop of the ISSMGE TC40 (Forensic Geotechnical Engineering) Hungary "Failures, Disputes, Causes and Solutions in Geotechnics"
Date: 24 - 25 September 2010
Location: (BME) 'A' Building , Budapest, Hungary
Organizer: TC40
• Contact person: Tensi Aviation Kft - Ms. Edit Hartung, Ms. Agnes Farago
• Address: 7621 Pécs, Teréz u. 17.
• Phone: 36 72 510 498, 513 983
• Fax: 36 72 510-497
• E-mail: afarago@tensipecs.hu, hartung.edit@tensipecs.hu
Website: issmge-tc40-hungary.net/main.php?menu=1

XX Argentinian Congress of Soil Mechanics and Geotechnical Engineering
Date: 6 - 9 October 2010
Location: CAMSIG 2010 , Capital, Mendoza, Argentina
Language: Spanish - English
Organizer: UTN - UNCu
• Contact person: Noemi Graciela Maldonado
• Address: Rodríguez 273
M5502AJE Capital
Mendoza
República Argentina
• Phone: 542615244572
• Fax: 542615244551
• E-mail: camsig2010@frm.utn.edu.ar
Website: www.frm.utn.edu.ar/camsig2010

DFI 35th Annual Conference on Deep Foundations
Date: 12 - 15 October 2010
Location: Renaissance Hollywood Hotel , Hollywood, CA, United States
Organizer: Deep Foundations Institute
• Contact person: Theresa Rappaport
• Address: 326 Lafayette Avenue
07506 Hawthorne, NJ
USA
• Phone: 9734234030
• Fax: 9734234031
• E-mail: trappaport@dfi.org
Website: www.deepfoundations2010.org

2nd International Conference on Geotechnical Engineering - ICGE 2010 - Innovative Geotechnical Engineering
Date: 25 - 27 October 2010
Location: Hammamet, Tunisia
Language: English and French
• Contact person: Dr Imen Said
• Address: National Engineering School of Tunis
ENIT, BP 37,

Event Diary (continued)

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Website:
www.enit.rnu.tn/fr/manifestations/icge2010/index.html

International Symposium on Geotechnical and Geosynthetics Engineering: Challenges and Opportunities on Climate Change
Date: 7 - 8 December 2010
Location: Miracle Grand Convention Hotel, Bangkok, Thailand
Language: English
Deadline for abstract submission: 1 August 2010
Organizer: Prof. Dennes T. Bergado
• Contact person: Conference Secretariat
• Address: c/o Asian Center for Soil Improvement and Geosynthetics, Asian Institute of Technology, P.O.Box 4, Klong Luang, Pathumthani 12120 Thailand
• Phone: +66-2-524-5500/12/23
• Fax: +66-2-524-6050
• E-mail: climatechange@ait.ac.th
Website: www.set.ait.ac.th/acsig/climatechange

4th International Conference on Geotechnical Engineering and Soil Mechanics
Date: 2 - 3 November 2010
Location: Power Institute of Technology, Tehran, Tehran, Iran
Language: English-Farsi
Organizer: Iranian Geotechnical Society
• Contact person: Dr. Ali Noorzad
• Address: Power and Water University of Technology
East Vafadar Boulevard
4th Tehran Pars Street,
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Tehran
Iran
• Phone: 98-21-7393-2487
• Fax: 98-21-7700-6660
• E-mail: noorzad@pwut.ac.ir

International Conference on Geotechnical Engineering
Date: 5 - 6 November 2010
Location: U.E.T. Lahore , Lahore, Pakistan
Language: English
Organizer: PGES & UET, Lahore
• Contact person: HAMID MASOOD QURESHI
• Address: GT&GE DIVISION, NESPAK HOUSE, 1-C, BLOCK N, MODEL TOWN EXTENSION
54700 LAHORE
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• Phone: 92-42-99090393
• Fax: 92-42-99231950
• E-mail: hamid833@hotmail.com, hamid.qureshi@nespak.com.pk

2nd International Symposium on Frontiers in Offshore Geotechnics (ISFOG)
Date: 8 - 10 November 2010
Location: Perth, Western Australia, Australia
Organizer: COFS
Secretary: • E-mail: ISFOG2010@civil.uwa.edu.au
Website: www.cofs.uwa.edu.au/ISFOG2010/

2011

Geo-Frontiers 2011
Date: 13 - 16 March 2011
Location: Sheraton Dallas Hotel , Dallas, Texas, United States
Language: English
Organizer: Geo-Institute
Secretary: • Contact person: Kristy Osman, Secretary General/Event Manager
• Phone: 1 651 225 6959
• E-mail: klosman@ifai.com
Website: www.geofrontiers11.com/index.cfm

Geotechnical Enginering for Disaster Prevention & Reduction
Date: 26 - 28 July 2011
Location: Fourth International Symposium , Khabarovsk, Russia
Language: English or Russian
Organizer: Far Eastern Transport Univ
• Contact person: Professor S.A.Kudryavtsev
• Address: Street Serishev, 47, Far Eastern State Transport University (FESTU)
680021 Kabarovsk
Russia
• Phone: 74212407540
• E-mail: its@festu.khv.ru
Website: www.igsh4.ru

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
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• Phone: 668190430 60
• Fax: 6625792265
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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

Event Diary (continued)

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FOR FURTHER DETAILS, PLEASE REFER TO THE ISSMGE WEBSITE
http://addon.webforum.com/issmge/index.asp

Editorial Remarks

The editorial board is pleased to send the ISSMGE members ISSMGE Bulletin Vol.4, Issue 2 in June 2010. The Editorial Board would like to thank all the members that contributed with articles for this issue. Any comments to improve the Bulletin are also welcome. Please contact a member of editorial board or Vice-President for the region, or directly e-mail to Prof. Ikuo Towhata, Chief Editor of ISSMGE Bulletin (towhata@geot.t.u-tokyo.ac.jp)

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The Foundation of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) was created to provide financial help to geo-engineers throughout the world who wish to further their geo-engineering knowledge and enhance their practice through various activities which they could not otherwise afford. These activities include attending conferences, participating in continuing education events, purchasing geotechnical reference books and manuals.

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