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Editorial Board

Message to ISSMGE from the President of the International Society for Rock Mechanics (ISRM)

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Dear Colleagues in the ISSMGE

There is currently a move to consider interaction between the international Societies concerned with soils and rocks, and so I am delighted to have been invited to contribute a message to you via the ISSMGE Bulletin. In fact, this is an ideal opportunity to ponder on the similarities and differences between our respective Societies and subjects and to consider some pertinent questions. There is less soil than rock in the world, so why is the ISSMGE larger than the ISRM? Although there are many subjects common to analysis methods for soils and rocks, e.g. the theory of elasticity, why are the engineering design approaches so different? Are the mechanisms of soil and rock failure different? Why is there so little interaction between researchers or practitioners in soil mechanics and rock mechanics? Why are there so few people fluent in the techniques of both soil and rock mechanics? In this message, I shall briefly explore these questions.

Why is the ISSMGE larger than the ISRM?
In 2009, the statistics of the two Societies were as listed in Table 1.

Table 1. Membership of the ISSMGE and the ISRM.

	ISSMGE	ISRM
Number of National Groups	86	48
Number of Individual Members	18,323	5,992
Number of Corporate Members	21	125
Number of Technical Committees/Commissions	24	9

Message to ISSMGE from the President of ISRM (continued)

John A Hudson

Apart from the 'anomaly' that the ISRM has many more Corporate Members than the ISSMGE, it is clear from Table 1 that the ISSMGE is a much larger organisation than the ISRM. Is this something which we would expect, or is it counter-intuitive? There is much more rock than soil in the world, and rock engineering projects reach much greater depths than soil engineering projects. Moreover, bearing in mind the vast volumes of rock that are mined, do we engineer more tonnes of rock than soil? On the other hand, many large cities are located on soil near river estuaries and foundation design in these conditions requires considerable soil behaviour understanding and detailed design work. Additionally, this urban concentration of soil mechanics engineers means that it is easier for them to get together — as compared to rock mechanics engineers who are found high in the mountains and deep in the earth where it is not so easy to congregate!

However, although the threefold disparity in size of the two Societies is somewhat of a mystery, I am pleased to say that the ISRM is currently in a steady membership growth period, as can be seen from Figure 1.

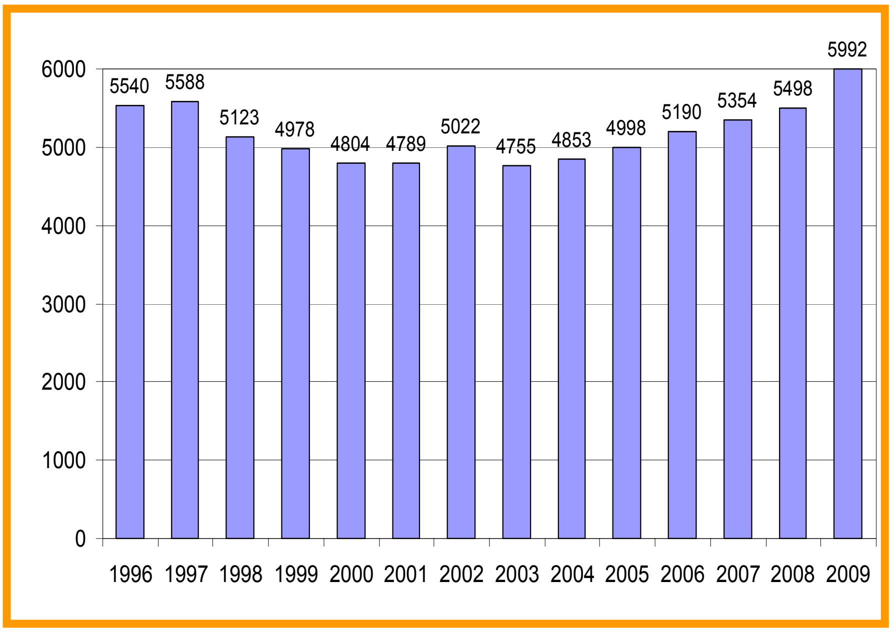


Figure 1. Individual membership of the ISRM, 1996-2009

Differences and similarities in the soil mechanics and rock mechanics subjects

<u>Pre-existing fractures</u>

One of the main differences in soil mechanics and rock mechanics design relates to block failure. When we are designing against failure in rock mechanics, there are two main modes of failure underground: rock block failure and stress-induced failure (Fig. 2). In soil mechanics, the soil particles are small compared to the size of the engineered structure but in rock mechanics the rock blocks, as generated by the natural fractures in the rock mass, can be smaller, of the same size, or larger than the engineered structure. This means that the pre-existing rock fractures have to be assessed in the site investigation to establish whether they can form rock blocks and, if so, whether these rock blocks fall or slide into the proposed excavation.

Message to ISSMGE from the President of ISRM (continued)

John A Hudson

The minimum number of faces that a rock block can have is four (a tetrahedral block); one of these faces can be the excavation surface, so that at least three fracture sets in the rock mass are required to generate the block. Given a knowledge of the fracture sets, computer programs are used to establish the likelihood of any such failure for a specific excavation geometry or to establish the optimal geometry to reduce the likelihood of any block instability. Wedge failure in an open-pit mine caused primarily by two faults is illustrated in Fig. 3.

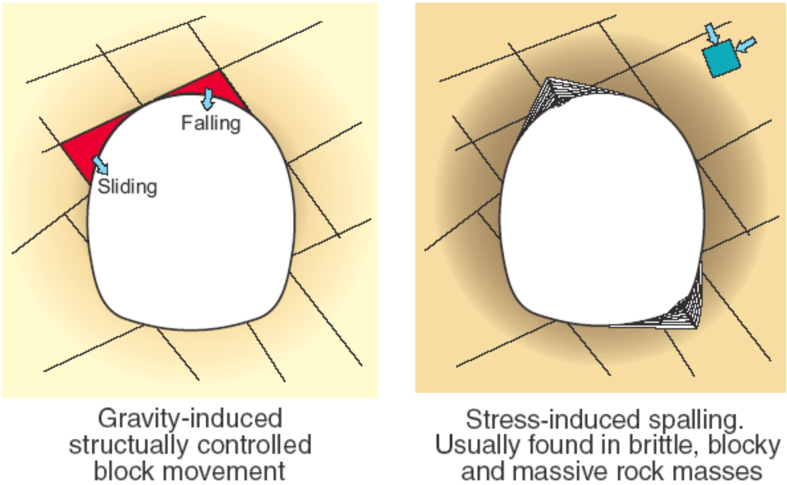


Figure 2. The two main modes of underground rock failure in hard rocks (from Prof Derek Martin).

Figure 3. Large wedge failure in the Teutonic Bore open-pit mine in Western Australia caused by the presence of two pre-existing major fractures.

In situ stress

Whilst there are many common factors in soil mechanics and rock mechanics, there are also significant differences. Another of these differences is the greater emphasis on *in situ* stress in rock mechanics. It is generally found in rock masses that the magnitude of the horizontal component of *in situ* rock stress is greater than that of the vertical component — caused by tectonic plate forces — and so it is useful and often critical to have a knowledge of the magnitudes and orientations of the principal stresses. Stress-induced rock spalling at the JinPing II hydroproject in China is illustrated in Fig. 4.

Message to ISSMGE from the President of ISRM (continued)

John A Hudson

Where there is flexibility in the orientation of rock caverns or tunnels, it is advantageous to orientate these in the direction of the maximum horizontal stress component in order to minimise stress-induced damage. The proposed design of an underground radioactive waste repository is shown in Fig. 5. This project involves many kilometres of tunnel and there is flexibility in their orientation.

Figure 4. Rock spalling in the tunnel shoulder at the JinPing II hydroproject on the Yalong River, Sichuan Province, China.

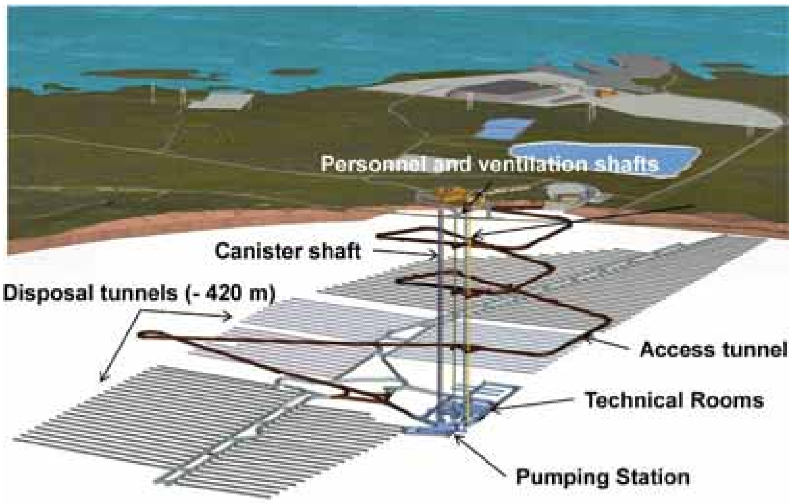


Figure 5. Orientation of radioactive waste repository tunnels in line with the direction of the major horizontal principal stress (from Posiva Oy, Finland).

The regional *in situ* stress trends can be obtained from the World Stress Map data but the local stresses can be perturbed from the regional trends and so it is necessary to measure the rock stress at the project site. However, this is not an easy task: all such stress measurement campaigns encounter some sort of problem. The reason for this is that the complete stress tensor with its six independent components cannot be measured directly. One either has to measure rock displacements after the stress has been removed (e.g. overcoring methods), or use fluid pressures (i.e. hydraulic testing methods), or use indirect methods (e.g. tests on borehole cores).

Message to ISSMGE from the President of ISRM (continued)

John A Hudson

The emphasis on *in situ* stresses in soil mechanics is not so strong, mainly because the soil engineering is conducted at shallower depths and because in soils there is not such a significant difference in the magnitudes of the two horizontal principal stresses.

Effective stresses and failure criteria

The concept of effective stress is well established in soil mechanics but is not so easily applied in a fractured rock mass where the water moves much faster along the fractures between the rock blocks than through the intact rock. The most commonly used failure criteria in rock mechanics are the Mohr-Coulomb and Hoek-Brown criteria. However, both of these only use two of the three principal stresses and so there is currently a thrust in rock mechanics to establish appropriate criteria containing the three principal stresses.

Design and numerical modelling

Numerical modelling now plays a major role in both soil and rock mechanics. The design flowchart in Fig. 6 indicates the eight main methods of design in rock mechanics. These eight methods increase in complexity from left to right (Methods A to D) and are in two rows of four, with the top row being 1:1 mapping methods (i.e. the geometry of the engineered structure is directly simulated in the model) and the lower row being non-1:1 mapping methods.

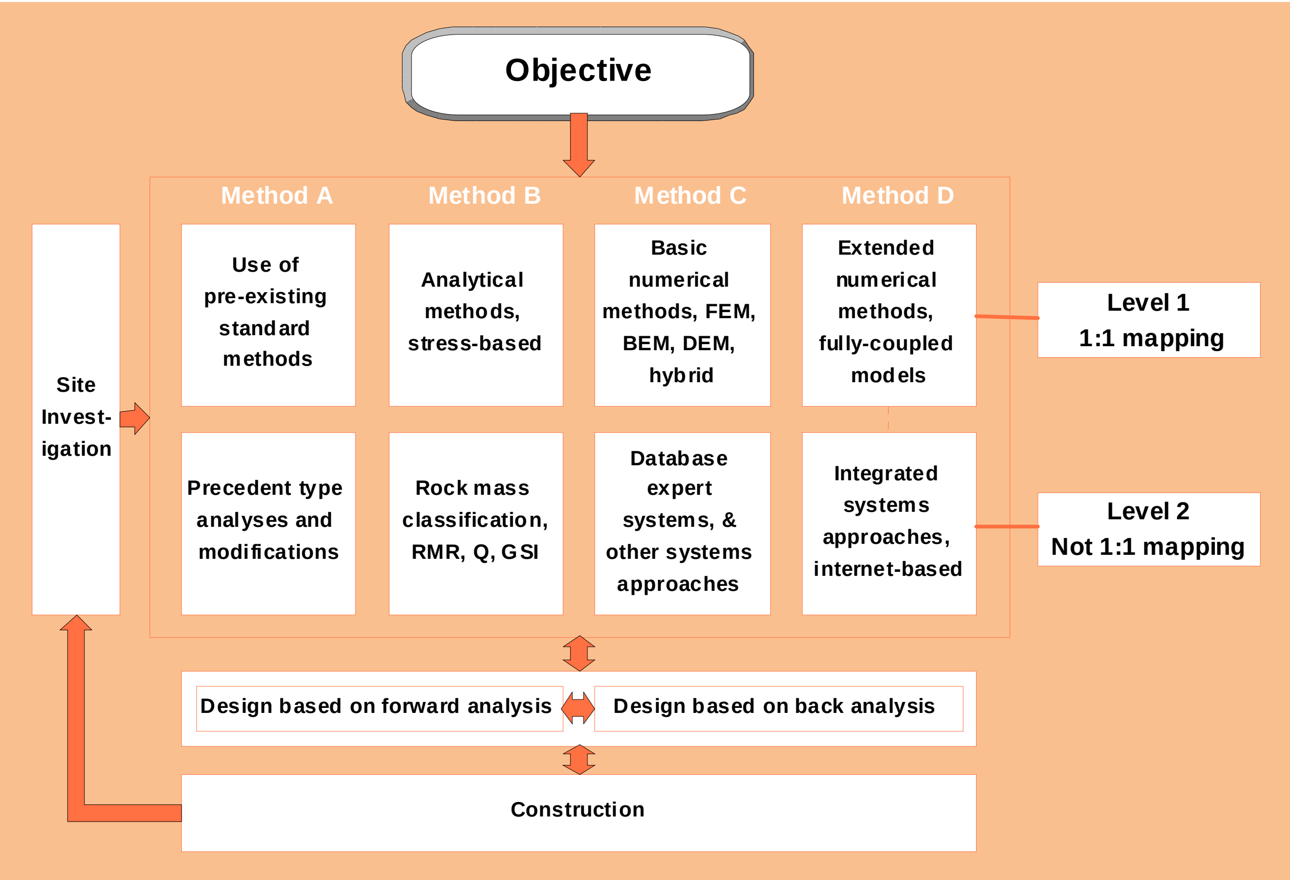


Figure 6. Design methods used in rock engineering.

Message to ISSMGE from the President of ISRM (continued)

John A Hudson

Whilst Methods A and B are successfully used for basic design approaches, there is a strong move to enhance the use of Methods C and D. The Method C techniques are now well established, but further research is required to develop the Method D techniques in which numerical methods can tackle design problems involving coupled thermo-hydro-mechanical-chemical systems and operate in association with internet-based approaches.

I have always been surprised that there is no international method of auditing the use of computer programs for rock engineering design and so one of our ISRM Commissions led by Professor Xia-Ting Feng of the Chinese Academy of Sciences (and our next ISRM President for the period 2011-2015) is developing an appropriate auditing capability.

Expertise in both soil and rock mechanics

Bearing in mind even just the subjects mentioned above, perhaps it is not surprising that few people are fluent in both soil and rock mechanics when there are so many differences in the respective approaches. On the other hand, there are cases where a knowledge of both subjects is required, for example for weathered rocks, foundations involving both soils and rocks, in mixed-face tunnelling, and cases where the ground can be regarded either as a stiff, strong soil or a soft, weak rock.

Potential Interactions between the ISSMGE and the ISRM

From the condensed discussion above, it is not immediately obvious how further interaction between the ISSMGE and the ISRM should progress. During graduate education, there are many common supporting subjects for students being trained in soil and rock mechanics, such as engineering geology, elasticity, visco-elasticity and plasticity. Additionally, familiarity with numerical techniques should apply equally to both subjects. Yet, somehow, the philosophies of approach, the techniques of site investigation and the specific design techniques in each subject are different in the final analyses.

Thus, on a generic educational, research or engineering basis, and bearing in mind all the points above, it is not clear to me how ISSMGE-ISRM interaction should develop, despite my feeling that such interaction is required and is overdue! If any readers of this Bulletin can see the way ahead, please let me know. Jean-Louis Briaud and I will be only too pleased to follow up on any constructive suggestions.

John A Hudson

London, UK

February, 2010

TC Activity

TC6 Committee on Unsaturated Soils

Recent past and future activities

The TC6 committee promotes cooperation and exchange of knowledge in the area of mechanics of unsaturated soils, including, as examples, expansive, collapsible, residual, and arid soils, and its relevance to the solution of engineering problems. Continuing from a rich history, the TC6 Committee on Unsaturated Soils has been busy the past two years and looks forward to exciting upcoming events. Some noteworthy recent events include the 4th Asian-Pacific Conference on Unsaturated Soils held in beautiful Newcastle, Australia in November, 2009 and most recently TC6 hosted a workshop at the 17th International Conference on Soil Mechanics and Geotechnical Engineering in Alexandria during September, 2009. The 3-hour long workshop on *Applications of Unsaturated Soil Mechanics in Geotechnical Engineering* included a prestigious slate of eight speakers from various countries including: Eduardo Alonso (Spain), Farimah Masrouri (France), Yu Jun Cui (France), Antonio Gens (Spain), Abdalla Harraz (Egypt), Tony Zhan (China), David Toll (UK), and Del Fredlund (Canada). Topics ranged from unsaturated soil behavior in freezing environments to desert conditions and applications ranging from high speed rail to landfills. By all accounts the workshop was well attended and informative.

Organisers and Contributors at the E-UNSAT 2008 Opening Ceremony (Durham, UK, June 2008)

(From left: Dr Domenico Gallipoli, Prof. Simon Wheeler, Prof. Neil Taylor (*Secretary General of ISSMGE*), Prof. Eduardo Alonso (*Chair of TC6*), Prof. Christopher Higgins (*Vice-Chancellor of Durham University*), Dr Charles Augarde and Dr David Toll)

TC Activity

TC6 Committee on Unsaturated Soils

Recent past and future activities (continued)

Still riding high from the successful UNSAT 2006 conference in Carefree, Arizona, TC6 is looking forward with great anticipation to UNSAT 2010 (5th International Conference on Unsaturated Soils) in Barcelona, Spain this September. The international UNSAT conference, which started in Paris in 1995, has become a 4-year recurring event and represents the crown jewel of TC6 conferences. Paper submissions and attendance have grown significantly since its inception, with about 250 papers being submitted for the UNSAT 2010 conference. For more information and to register for this exciting conference in Barcelona, refer to the web site (http://congress.cimne.com/unsat2010) – be sure to take advantage of the early bird registration by April 30. At the well attended 1st European Conference on Unsaturated Soils in Durham, UK, in June of 2008, the TC6 committee met and after extensive deliberations selected Brisbane, Australia for the UNSAT 2014 venue. Other significant upcoming events include the 5th Asian-Pacific Conference on Unsaturated Soils, which will be held in Pattaya, Thailand in 2011 and the 2nd European Conference on Unsaturated Soils with possible venues in Germany or Italy in 2012.

In addition to conference and workshop activities, the TC6 web site continues to be maintained and well-utilized. The Unsaturated Soils web site can be accessed at http://www.dur.ac.uk/geo-engineering/unsaturated/tc6/tc6.html. The web site includes information on upcoming events, publications, and communications of interest to those in the field of unsaturated soil mechanics.

Delegates at the E-UNSAT 2008 conference

Case History

The Pinnacle Tower – Geotechnical Challenges

Dinesh Patel, Sarah Glover, Jonathan Chew, Jenny Austin
Ove Arup and Partners, London, United Kingdom

The Pinnacle Tower is one of a cluster of towers being constructed in the heart of the City of London. When complete it will be 62 Storeys high – taller than any other building in the UK. The design and construction of the tallest building in the UK, on a central London site occupied by 3 existing buildings presented special challenges. The approach the design and construction team took to these challenges earned them the best geotechnical project over £1M at the recent GE awards.

The new development

The new building is to be 62 storeys with a 3 level basement (Figure 1) occupying a retail and commercial office space of about 1.4M sq ft. Demolition of the previous 10 storey buildings to ground level started in mid 2007. Pile construction started in July 2008 from the ground level slab, over an existing three level basement which occupied much of the site. Piling is now complete, demolition and basement excavation continues as with pile cap construction progresses. The new substructure and superstructure is planned for completion in 2012.

Figure 1: The Pinnacle Tower (copyright KPF web site)

In London, tall buildings are typically less than 200m high and have traditionally been founded on large diameter bored piles (including under-reamed piles) in London Clay typically 25 to 35m deep. Canary Wharf Tower at 235m, is founded characteristically on 25m deep base grouted bored piles in the Thanet Sand, supporting maximum loads of about 30MN. The Pinnacle, has typical column loads up to 45MN with some extreme loads of up to 70MN and cannot be supported on any currently known piling system drilling into just London Clay. For this reason the only sensible solution was to found into the Thanet Sand, which at this site is about 63m below street level. The very high loads resulted in piles having diameters up to 2.4m.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

Most of the experience on base grouted piles in Thanet Sand has come from projects at Canary Wharf and therefore there is very limited experience of piling larger than 1.8m diameter, into Thanet Sand, within the City of London. Base grouted Thanet Sand piles at The Pinnacle were a much greater diameter and depth than any experience gained from past projects using base grouted piles. This posed significant design and construction challenges, which are described in this paper.

The project team and its organisation are detailed in Figure 2.

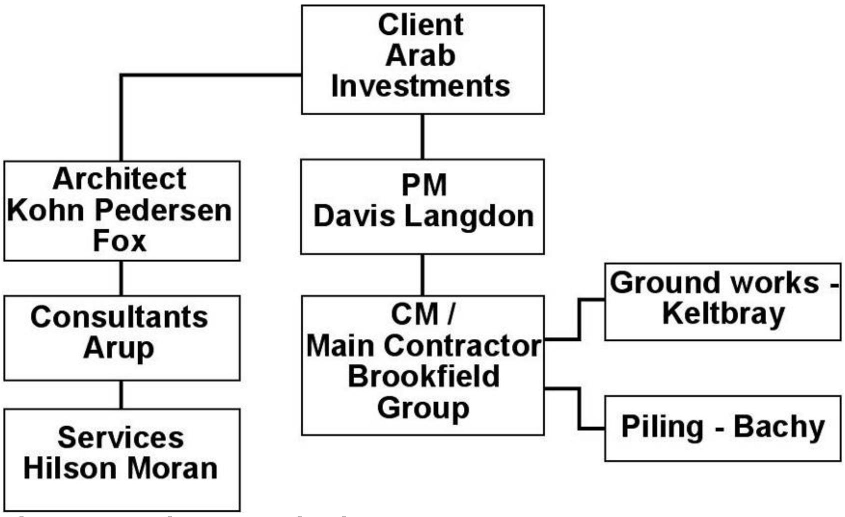


Figure 2: Project Organisation

The Site

The plan area of the site is approximately 140m by 70m and covers the footprint of three previous buildings. Surrounding street levels are approximately +16.8mOD on the west falling to +15.4mOD on the east side. Previous buildings on the site had a 14m deep basement (38 Bishopsgate, cc 1985), a 12m deep basement (22-24 Bishopsgate, cc 1975) and a single level basement for the oldest structure (4 Crosby Court, cc 1908). The largest of the three is Standard Chartered Bank, which was founded on up to 1.5m diameter bored piles with underream bells at 35m below ground level 4.5m in diameter in the London Clay. The existing base slab is a minimum 1m deep raft, in places up to 2.5m thick.

The footprint of The Pinnacle covers all three buildings with one tower leg sitting in the pavement of Bishopsgate. There will be a common 14m deep basement across the whole of the site formed within the existing basement walls and a new secant pile wall to be constructed in the southeast of the site underneath Crosby Square where there is currently no basement and Crosby Court where there is only a single level basement

The existing Standard Chartered Bank Building already has a basement at about +3.0mOD, and the new base slab will be 0.8m thick to be cast on top of this old slab. The new base slab will extend to the other parts of the site which have been excavated to the same level.

The Pinnacle abuts two buildings, a 24 storey structure (6-8 Bishopsgate) founded on a piled raft and 1 Great St Helens, a 10 storey structure founded on a mini-piled raft. Immediately to the south east there is also 122 Leadenhall, the site of a future 225m tall tower (Figure 3).

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

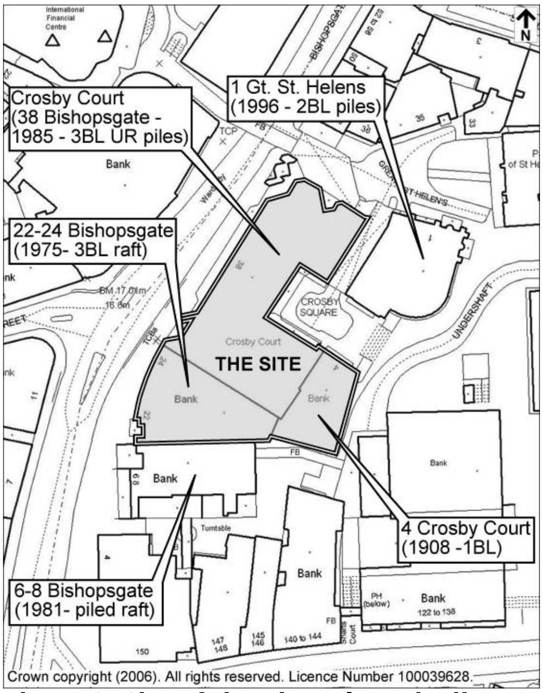


Figure 3: Site of the Pinnacle and adjacent buildings (BL = no of basement levels)

Figure 4: Anticipated layout of underream piles based on original plans

Mitigation of ground risk and challenges

Old foundations

Major project risks were posed from foundations of the previous developments and temporary piles used to form the original basement the extent and nature of which were not fully understood. It was important to establish the extent and location of these old foundations and temporary works to minimise adverse impact on the project and to understand better whether any benefit could be gained through foundation re-use. Arup undertook rigorous research and consultation of various consultants, previous main contractors of the existing buildings, piling contractors and Building Control departments were consulted to obtain old design and construction records.

The previous consultant's tender information was useful as they recorded structural loads and underream piles. Temporary works construction sequences and plans for a 14m deep bottom up basement construction were also obtained. The contractor also provided photographs of the basement construction which clearly showed that there would be a large number of temporary king post piles in the ground, which were utilised in 1985 to support the basement construction – these could obstruct the construction of new piles.

Unfortunately, founding levels of the existing foundations had not been recorded though indicative pile toe levels of -20.0mOD were given for some piles. No definitive design or pile records were available. The best estimates of the position of these underream piles and sizes are given on Figure 4. Intrusive investigation was required to confirm the locations and founding levels of the existing piles.

From talking to the original contractor of the Standard Chartered Bank, it was revealed that the northern half of the site had only short (6m deep) reinforcement cages installed below the trim level of the underream piles. It was found during basement excavation in 1985 that all these piles cracked just below the reinforcement cage, and before the building could be built these piles had to be remediated by coring through the underream piles and grouting to close the cracks.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

Without the as-built sizes of the shafts and under-ream bases the cost and time associated with concrete coring during piling works was still considered a major risk and would have a severe impact on the overall piling programme. To minimise this risk, early in the design process, a probing and coring exercise was carried out to investigate the location, depth and size of the undeream bases and the strength of the concrete. This used low headroom Fondedile (now Keller Geotechnique) piling rig, operating within the basement and while the building was still occupied (Figure 5). An investigation of the RC perimeter walls, including old diaphragm walls, was also made. Concrete cores and strength tests allowed the temporary works contractor to design his propping scheme for the existing basement demolition and the piling contractor to determine the most appropriate coring equipment/piling rig.

This information allowed the designers to mitigate the piling risk at an early stage of the design, by minimising the number of new piles that would encounter the existing piles. The pile removal costs and programme implications could then be more accurately assessed before tendering the main contract. This was extremely important as the piling was to be carried out from ground slab level through three floors of existing basement. Therefore adjusting new pile locations once piling works has started was not an option and would have been prohibitively expensive, time consuming and disruptive.

Figure 5: Probing and coring of existing UR piles

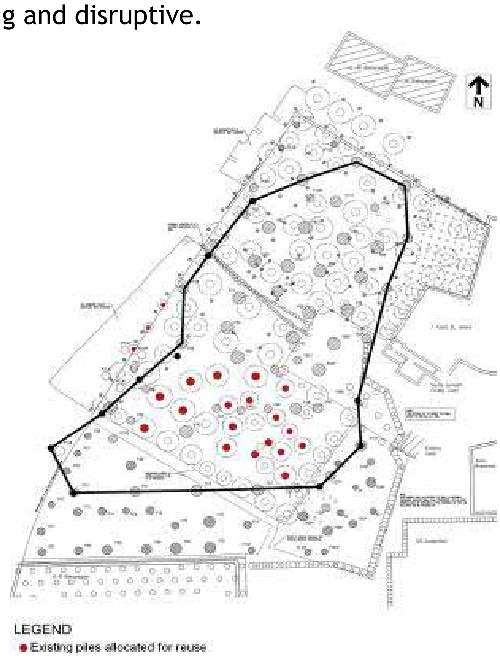


Figure 6: New pile layout and megaframe piles (also shown, extended basement within new secant pile wall to southeast)

The foundation scheme

The new foundation layout (Figure 6) also posed an engineering challenge. The outline of the megaframe, which is a fundamental part of the superstructure stability system, was spatially curved and twisted and did not lend itself to foundation reuse. Some megaframe columns also landed close to the boundary of the perimeter walls with one column sited in the street. Early design schemes considered reusing all the 1985 underream piles to support the new internal podium and substructures. All the piles in the southern half of the site were fully reinforced over the shaft and did not suffer from the cracking that the northern piles had. The cracks in the northernmost piles could be expected to reopen due to an average short term net unloading of the site of about 200kPa. As a consequence, only the southern half of the site could be considered for foundation reuse. A mixture of new large diameter bored piles and minipiles, founded in London Clay, were used in the northern half of the site, see Figure 6.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

The megaframe column loads were between 20MN and 45MN. At early design stage, the use of pairs of 1.8m diameter piles per column was considered as an alternative to the single large diameter piles per column solution. However, this would have substantially increased shaft and base coring of existing underream piles (some shafts were fully reinforced), required more drilling out of steel king post obstructions, and more pre-drilling of the existing thick base slab. It would also have introduced significant pile caps and hence larger openings in the existing base slab. The Construction Managers also wanted to start superstructure construction off the pile heads whilst demolishing the basement top down, therefore large pile caps would have been time consuming to construct and would have delayed this programme. A piled raft solution was also considered but again the perimeter megaframe carried the majority of the structural loads and this frame was not sympathetic to such a foundation solution. Therefore a decision was taken to found the structure on single piles per column, using 1.5m to 2.4m diameter base grouted piles founded about 65m into the Thanet Sand stratum.

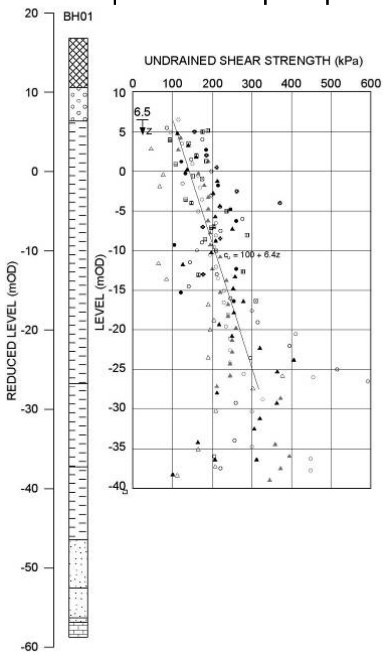
Ground conditions

The Pinnacle site stratigraphy is summed up in Table 1:

Table 1: Stratigraphy at The Pinnacle site

Stratum	Thickness (m)
Made Ground	6
Brickearth/ River Terrace Gravels	4.5
London Clay	35
Lambeth Group	18
Thanet Sand	11
Chalk	2 (proven)

The clays are underdrained due to the low groundwater table in the lower aquifer of the Chalk in Central London. A summary of the stratigraphy of the site with undrained shear strength is also plotted on Figure 7a. The piezometric pore pressure is given in Figure 7b.



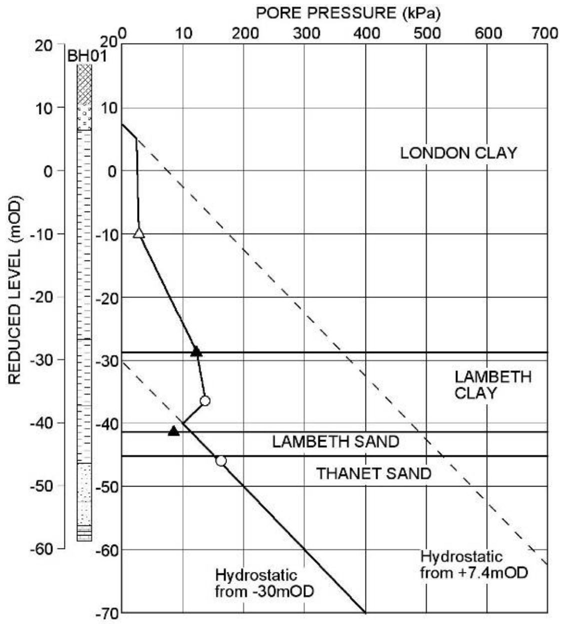


Figure 7 Ground Investigation data: (a) Undrained Shear strength profile; (b) Piezometric pressures

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

Arup experience at Canary Wharf showed that it was important to understand the mineralogy of the Thanet Sand, as low bearing capacities could occur in sand with high clay content. For this reason, the site investigation at The Pinnacle considered profiling of the Thanet Sand as a crucial part of the pile design. This was achieved by carrying out Ménard Pressuremeter tests (Figure 8a) and frequent pipette / sieve analysis of the Thanet Sand from high quality rotary cored samples (Figure 8b).

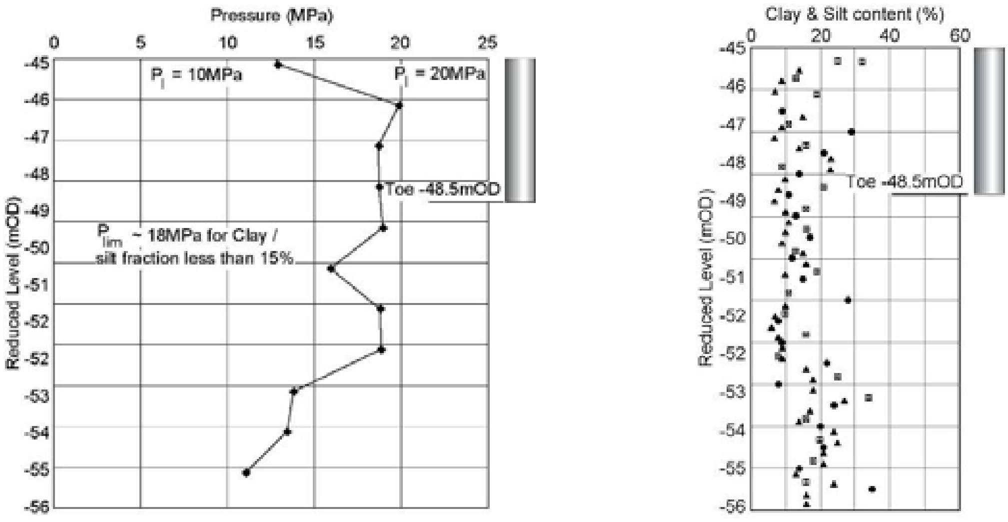


Figure 8: (a) Limiting pressures from Ménard pressuremeter in Thanet Sand; (b) Clay /silt fraction (%) of Thanet Sand

The results of these tests show that the upper 7m of the Thanet Sand recorded high limiting pressures (19MPa) and correspondingly low clay/silt mineralogy of less than 15% (referred to as “clean” sand). The lower 4m of the sand, referred to as “dirty” sand, has a higher mineralogy (> 20%) and limiting pressures reduce to about 11MPa. These observations are similar to the conclusions made by Nicholson et al (2002) in the Thanet Sands at Canary Wharf. From this investigation, the decision was taken to found all The Pinnacle base grouted piles at least 2m below the sand surface at -48.5mOD. The risk of piles not founding into the Thanet Sand, due to varying surface levels across the site, was not considered a major risk as there was a clear marker bed, the Pebble Beds of the Upnor Formation, separating the interface between the Lambeth Group and the Thanet Sand.

Pushing boundaries – design of base grouted piles

The first major use of base grouted piles in Thanet Sand was developed for the buildings at Canary Wharf, Docklands in the 1980’s (Troughton 1989). This form of piling was well suited to these sites as the Thanet Sand was only about 30m below ground with mean effective stresses at about 300 – 400kPa. Two methods of pile design evolved on these sites, one based on effective stress design, and the second using self boring and/or Ménard Pressuremeter testing as described in more recent works (Chapman et al 1999, Nicholson et al 2002). The base grouted piles at Canary Wharf did not exceed 1.8m diameter and base grouting was carried out using a maximum of four grouting tube circuits or tube à manchettes (i.e. using 8 grouting tubes) attached to a reinforcement cage. It was also possible to build these piles generally within 12- 24hrs. Maximum loads on the piles were 32MN, assuming a working stress of 12.5MPa. The early Thanet Sand piles at Canary Wharf were constructed under bentonite, but later projects were constructed in dry Thanet Sand, as it was dewatered for basement construction. The use of pile drilling augers and digging buckets under bentonite was thought to loosen the Thanet Sand, and base grouting was employed to restore the base stiffness (Yates and O’Riordan, 1989).

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

In the City of London, there is very limited experience of piling up to 65m into saturated Thanet Sand. The Moorhouse development (Yeow et al, 2005) is the closet site to The Pinnacle where 1.8m diameter base grouted piles were formed supporting a maximum of 35MN column loads. Pile loads on The Pinnacle are significantly higher, up to 45MN, meaning up to 2.4m diameter piles founded 65m below ground are required. Therefore this posed challenges to both the design and construction, which is discussed below.

Early workshops with potential piling contractors indicated that piles of this size and depth would take about 4 days to build, even longer if shaft or base coring of underream piles had to be carried out - much longer than at Moorhouse. As a result there was concern that low shaft frictions may occur in the overlying London Clay and Lambeth Clay strata, compared with piles which typically took 12-24hrs to install.

The pile design strategy developed for The Pinnacle therefore had to consider all these risks and the final design is illustrated in Figure 9.

Initial design was carried out using experience gained at Canary Wharf and Moorhouse, where the end bearing capacity factor Nq*, ranged between 30 and 60, from pile testing. At The Pinnacle site, the mean effective stress at the pile toe is about 800 kPa, and using the lower Nq* factor results in an ultimate base stress of about 24MPa. Thus, piles would be limited by the working stress on the concrete for an overall factor of safety of 2.5. Base grouted piles, which support the megaframe column loads, range between 1.5m and 2.4m diameter. For the largest pile diameters, up to 50% of the working load is carried on the grouted base.

An alternative design approach is to calculate the ultimate base stress from the limiting pressure derived from the Ménard Pressuremeter test, using the approach below:

$q_b = \lambda .\ p_{lim}$

where
λ = factor to convert the Ménard limiting pressure to end bearing coefficient
p_{lim} = limiting pressure from Ménard pressuremeter

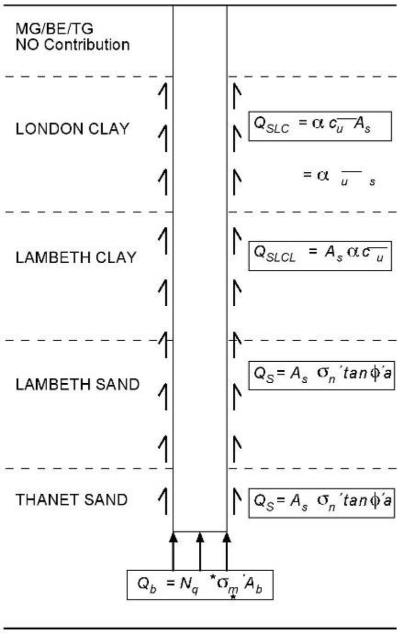
For this design approach, preliminary pile tests would have to be carried out to select an appropriate λ value.

Heave and settlement

Heave forces were generated in all the piles due to (a) demolition of the existing buildings which weighed between 80kPa and 260kPa (b) areas of the site where new basements up to 14m were to be excavated (e.g. 4 Crosby Court) and (c) heave induced swelling pressures acting under the ground bearing slab due to long term changes in pore pressures. A simplified approach modifying the work of O'Reilly et al (1990) was used to calculate the heave forces in the new base grouted piles, London Clay piles and minipiles, and full length reinforcement was placed to avoid cracking in the piles. A 3D Finite Element model of the basement (Figure 10) was also carried out to check this design, to assess ground movements, the impact on adjacent buildings, the raft design and to investigate the response of the tower to soil-structure interaction.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)



Where,

α = ranges between 0.2 to 0.4
a = ranges between 0.45 and 0.7
$N_q{}^*$ = 30
F = 2.5 overall
Limiting working stress on base is 10MPa

Figure 9: Pile Design Methodology

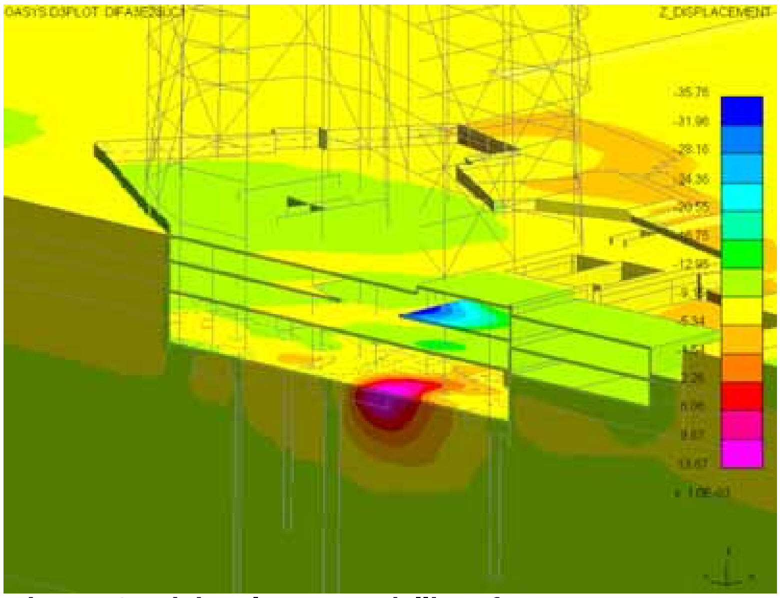


Figure 10: Finite Element Modelling of Long Term Ground Movements (Vertical Displacements)

Preliminary test pile

A 900mm diameter, 64m long preliminary pile test was carried out at The Pinnacle site to confirm the design parameters for the base. The pile was double sleeved to about 11m into the London Clay and single sleeved with a bitumen coated permanent casing to about 51m (-35mOD), approximately 7m into the Lambeth Group, to eliminate the shaft resistance of overlying soils. Figure 11 shows the test pile instrumentation layout.

Incremental maintained load testing to about 25MN took place 14 days after casting the pile. The load settlement performance of the test pile is shown on Figure 12. Also, shown is the test conducted on a similar size pile and embedment length (56m) at Moorhouse (Yeow et al). Both piles exhibit similar load settlement behaviour. The ultimate pile capacity was not achieved in the third cycle of loading to 24.5MN where the pile head settlement was 105mm. About 60% of this movement was due to elastic shortening of the pile.

The extrapolated ultimate pile capacity was determined from Fleming's (1992) analysis, and also checked using the simplified Chin (1970) method, which as expected gave a slightly higher capacity. The Fleming method predicted an ultimate base capacity of 29MN and an ultimate base stress of 45.6MPa was deduced. The backfigured end bearing capacity factor was deduced as N_q * = 57 and this value compares favourably with the N_q * = 60 derived from the Moorhouse pile test.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

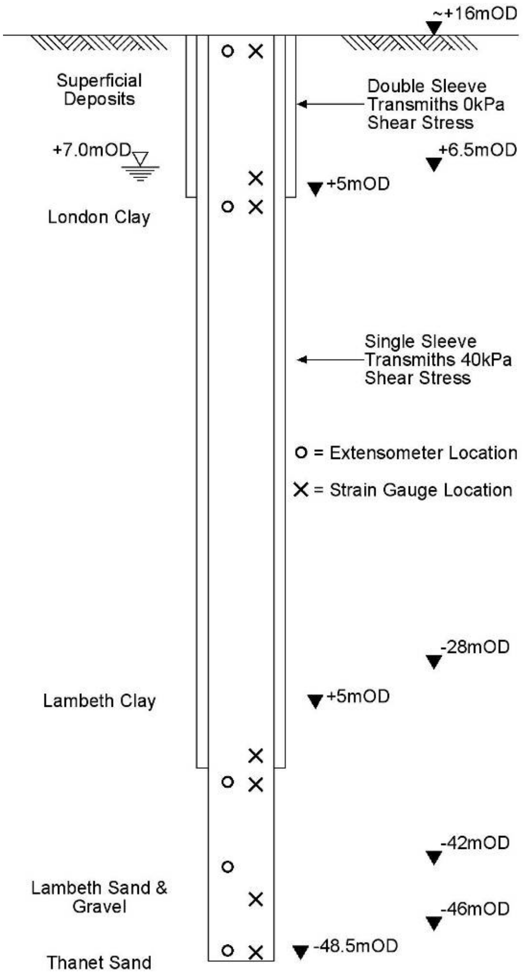


Figure 11: Pinnacle Test pile instrumentation

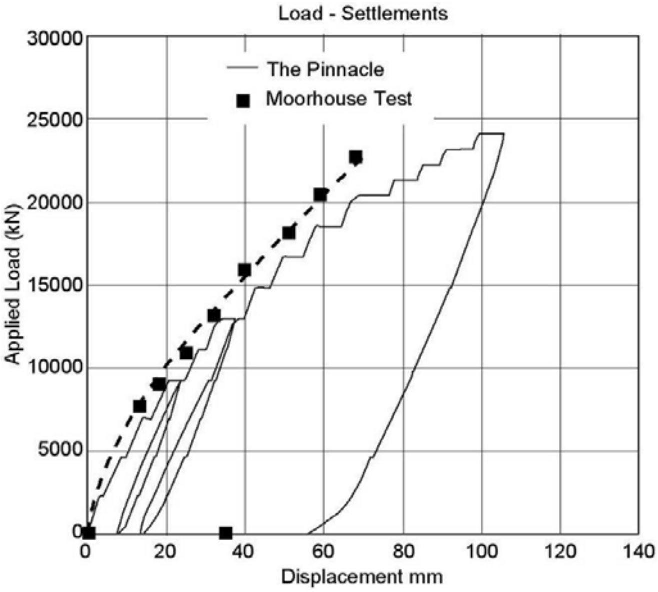


Figure 12: Load settlement plot of test pile

These two pile test data were also compared with the results of a large number of recent pile tests carried out by Arup at Canary Wharf on up to 1.5m diameter piles embedded at different depths in the Thanet Sand, see Figure 13. At less than 5m embedment the N_q* is greater than 60, while below 9m, N_q* is less than 35. Both the Pinnacle and Moorhouse test data in the City, are piles with less than 3m embedment and show high end bearing capacity factors, $N_q^* \geq 57$, consistent with the Canary Wharf results (circled in Figure 13). Nicholson et al 2002, showed that the reduced N_q* factors with increasing embedment depth was due to an increase in clay minerals in the lower part of the Thanet Sand at Canary Wharf.

Using the Ménard Pressuremeter and a limiting pressure, p_{lim} = 19MPa, a λ factor of 2.4 was deduced from the extrapolated ultimate base capacity.

In summary, good quality zoning of the Thanet Sand is important for pile design and this can be easily done using the Ménard pressuremeter plus taking frequent pipette/sieve analysis of the Thanet Sand.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

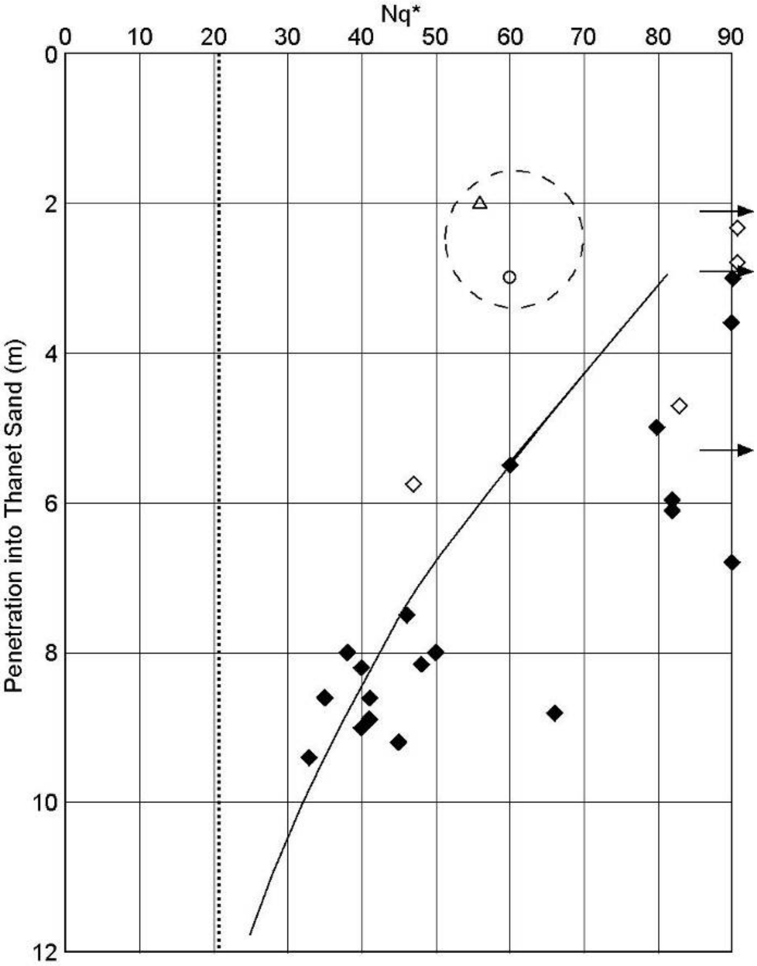


Figure 13: Base Factor, N_q* from base grouted test piles in the City and Canary Wharf

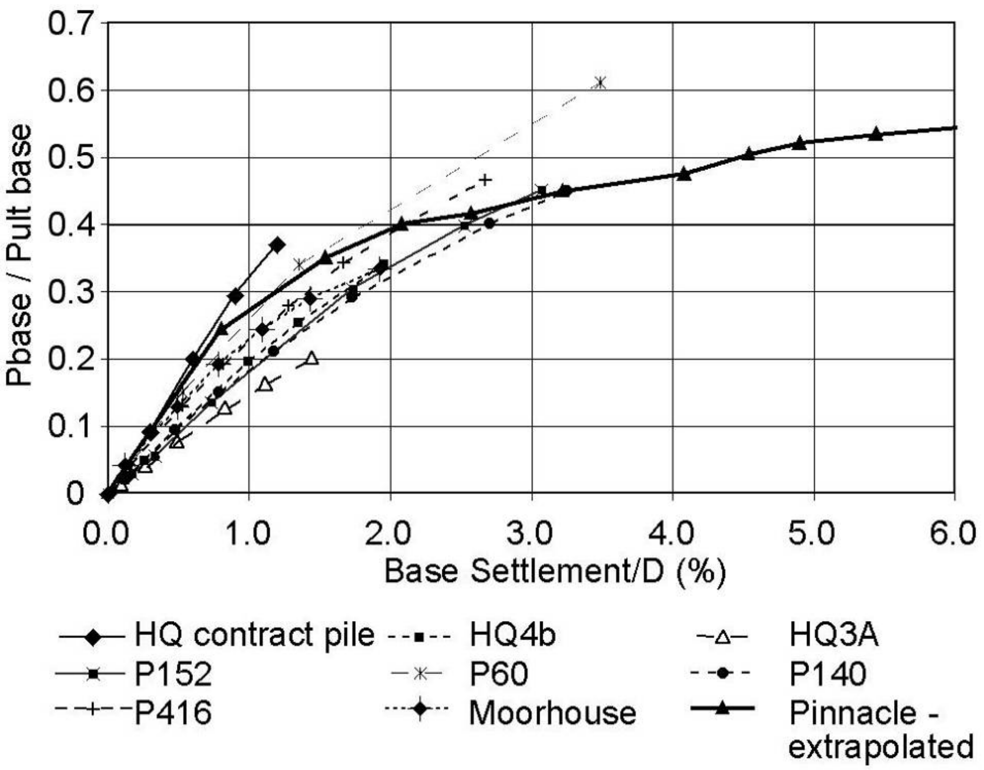
Estimating base settlement

The load-settlement curves of preliminary pile tests at Canary Wharf, The Pinnacle and Moorhouse were studied and a normalised base load against normalised settlement plot produced, as shown in Figure 14. These represent load tests conducted on pile sizes from 0.9 to 1.5m diameter. The applied load on the base can be calculated from the working load (WL) and the ultimate base capacity from $Q_b = N_q^*. \sigma'_m . A_b$.

The total pile settlement can then be deduced, using Figure 14 to estimate the base settlement and adding this to the calculated elastic shortening of the pile concrete. This approach was used to calculate the individual settlement of the base grouted piles for The Pinnacle at the end of the construction phase. Numerical techniques were used to calculate the long term settlements of the building, due to changes in pore pressures in the London Clay caused by unloading and reloading of the site, and pile group effects.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)



Definitions:
P_{base} - load applied on base ; $P_{ult\ base}$ – ultimate base capacity (Q_b) ; D – Base diameter

Figure 14: Normalised base load and settlement plot

Construction Issues
The construction of the base grouted piles in a congested City site posed many engineering challenges. The major risks associated with potential softening of the shaft and in particular the base was:

- not leaving a soil plug before final cleaning;
- leaving pile bores open for too long between cleaning the base and concreting;
- disturbing the base unnecessarily after base cleaning;
- not carrying out the proper checks on the base stiffness; and
- not controlling the quality of bentonite.

These risks were all judged to be unacceptable for self certification by a piling contractor and therefore Arup had two Resident Engineers (day shift and night shift) monitoring the works and alerting Arup designers to any potential problems on site. This proved to be important as a close working relationship between Arup and Bachy allowed piling construction problems to be quickly resolved on site, minimising delays.

At scheme design stage various options for basement demolition and propping were considered. In addition a decision had to be taken on whether piling should commence from base slab level or at ground level. The Construction Managers opted for piling from existing ground floor slab level (+16.5mOD) once the buildings had been demolished.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

This meant that the Bauer BG40 piling rig sat on the ground floor slab above three levels of basement that were temporarily back propped, as shown in Figure 15. Holes were cut through all the basement levels and the base slab to allow 18m deep temporary casings to be installed ready for piling.

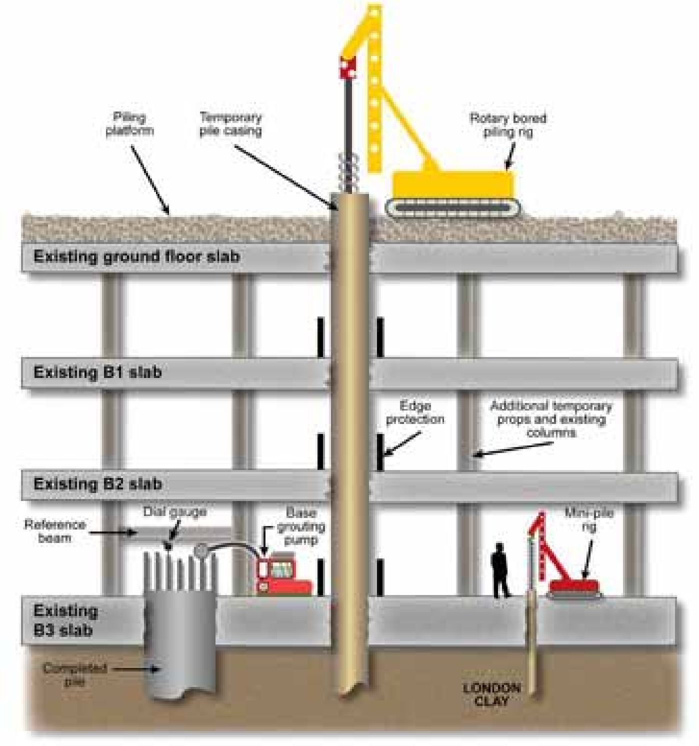


Figure 15: Piling from existing basement ground slab

Piles were constructed over two working shifts per day (working 17hrs out of 24hrs) and each base grouted pile took at least 2 days to construct, provided there was no shaft and base coring of old piles.

Some piles had piling cut-off levels close to the existing B3 slab, in which case the bentonite polluted concrete was brought up into a temporary formwork box constructed at B3 level (Figure 15) to allow the temporary casing to be removed. The polluted concrete was carefully removed, 1-2 days later, with an excavator bucket. All the piles were installed with grout tubes (doubling as sonic logging tubes) and extra pairs of extensometer tubes were installed with the pile cages to measure pile toe uplift, during base grouting.

Tube à manchette (TAM) arrangements
Previous experience with successful base grouting of piles was for piles up to 1.8m diameter, where 4 grout circuits (8 tubes total) are used (Figure 16). The maximum grout area potentially covered by these circuits is about 0.6m^2, and the maximum grout reach is about 0.6m to the pile centre.

To ensure the same coverage of grout area and reach, the 2.1m diameter piles were fitted with 6 TAM's, and the 2.4m diameter piles with 8 TAM's, as shown on Figure 16. A second reinforcing cage was avoided by 'joggling' four of the outer 8 TAMs into the centre of the pile over the bottom 10m, as shown in Figure 16. The outer 4 TAM circuits continued to the base and were also used to sonic log each pile before grouting.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

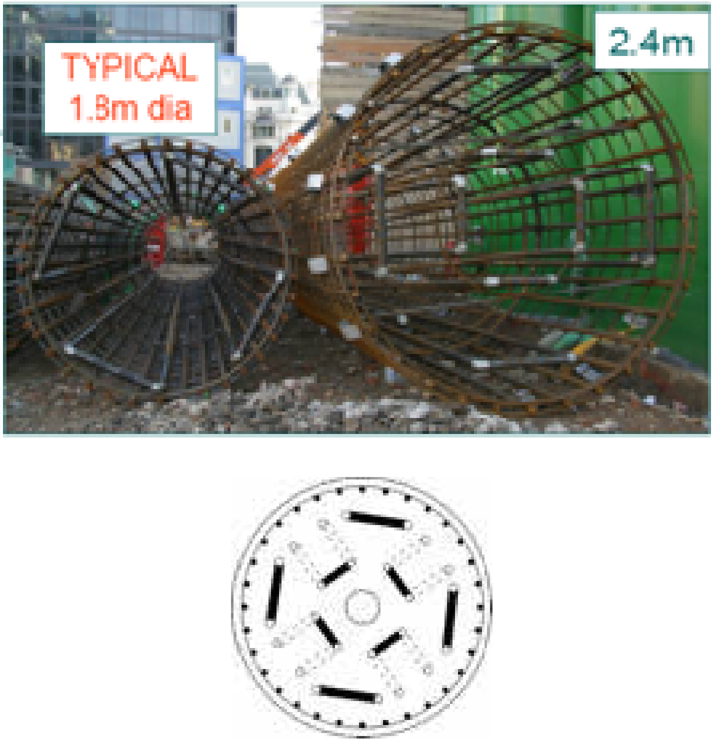


Figure 16: TAM arrangements for 1.8m and 2.4m diameter piles and sketch showing 'joggle'

Bentonite Quality Control
The piling contractor tightly controlled the quality of the bentonite through all stages of the piling and Table 2 shows the results of these site tests against the specification limits, as given in BS EN 1538:2000.

Table 2: BS EN 1538:2000 Bentonite Limits

Property	Stages			
	Fresh	Ready for re-use	Before concre-ting	Site Results
Density (g/ml)	<1.10	<1.25	<1.15	**1.05 - 1.1**
Marsh Value (s)	32 to 50	32 to 60	32 to 50	**30-40**
Fluid Loss (ml)	<30	<50	N/A	**14-18**
pH	7-11	1-12	N/A	**As spec**
Sand Content	N/A	N/A	<4	**0.25 - 2.0**
Filter Cake (mm)	<3	< 6	N/A	**<1**
After BS EN 1538: 2000 Table1 - Execution of Special Geotechnical Works - DWs				

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

Base Cleaning and Ranking System for base grouted piles
A non-circular flat bladed cleaning bucket was used to cut the final 150mm of the pile base. This special bucket reduced the suctions generated at the base during bucket extraction and it also had holes on the side so that bentonite fluid could runaway. Following this operation, a 100mm square metal plate attached to a length of fabric tape measure was lowered to the base of the pile to check the base hardness. This was done immediately after base cleaning and after installation of the cage (just prior to concreting). The last measurement was taken to score the base hardness using a ranking system developed specifically for The Pinnacle site, see Table 3.

Scores were given for an OK, Firm or Hard base by the Resident Engineer. In the same way marks for grout pressures, grout take and base uplift (measured with extensometers located at the pile toe), were combined to give an overall ranking score. Grout volume was estimated for each pile size and ranked according to the actual grout take. If the peak grout pressure was less than 30 bar on each circuit during the first grouting phase then the pile was to be regrouted, up to a limit of three grouting operations including the initial operation. A minimum of three hours was allowed between each round of grouting. Using this ranking system a minimum score 7 out of a possible 15 was considered as an acceptable pile.

Conclusions
The Pinnacle is founded on some of the deepest and largest diameter base grouted piles ever built in the City of London. Single piles support loads of up to 45MN and were built through three levels of existing basement in a congested city site. The base grouted piles also had to be drilled through the bases (sometimes the shafts) of underream piles from the previous development, as well as king posts from earlier temporary works left in the ground. This made the construction very challenging. A preliminary instrumented pile test to about 25MN was carried out to confirm the design parameters. The results of this test showed good correlation with design of base grouted piles at the Moorhouse and Canary Wharf sites. Rigorous site controls were implemented checking the quality of the bentonite, base stiffness, grout volume and pressure, and base uplift for all the base grouted piles, using a ranking system that was specially developed for this site.

Acknowledgements
The success of the construction work at Pinnacle could only be achieved through innovation and collaboration between the design and construction teams. Accordingly the authors wish to acknowledge Soil Mechanics, Bachy Soletanche, Mace and Brookfields team for their contribution and co-operation throughout the project.

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Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

Table 3: Base grouted pile ranking system

Parameter	Ranking System Criteria					Rank
1. Base uplift	>0.15 mm					3
	0.1 - 0.15 mm					2
	<0.1 mm					0
2. Base Stiffness	Hard (Grade I)					3
	Firm (Grade II)					2
	OK (Grade III)					1
	No information/soft/very soft (Grades IV & V)					-2
3. Grout volume, litres	Pile diameter (m)					
	1.2	1.5	1.8	2.1	2.4	
	>150	>200	>300	>400	>500	3
	100 - 150	150 - 200	225 - 300	300 -400	375 - 500	2
	75 - 100	100 - 150	150 - 225	200 - 300	250 - 375	1
	<75	<100	<150	<200	<250	-1
4. Max grout pressure (mean on final phase), bar	>70 bar					3
	40 - 70 bar					1
	30 - 40 bar					0
	<30 bar					-2
5. Av. Residual pressure (mean on final phase), bar	>30 bar					3
	20 - 30 bar					2
	15 - 20 bar					0
	<15 bar					-2

Notes:

1. The pile base stiffness is to be measured at the following times during boring:
   - Directly after base cleaning. A minimum stiffness of 'OK' must be achieved at this stage.
   - After reinforcement cage installation.
   - If the time between completion of cage installation and start of concreting is greater than 1 hour then the pile base shall be rechecked immediately prior to placing the concrete.

   The last measurement taken shall be used for the score in the ranking system.
2. Pressure measurements for peak and residual pressure criteria will only be accepted if a minimum volume of 5 litres of grout (after allowing for the compliance of the delivery system) has been injected.
3. If a peak pressure of 30bar and residual pressure of 15bar are not achieved on each circuit during the first grouting phase then the pile shall be re-grouted, up to a limit of three grouting operations including the initial operation. A minimum of 3 hours must be allowed between each round of grouting.
4. Pile head uplift shall not exceed 2mm.
5. A maximum of three grouting operations (including the initial operation) shall be carried out.

Case History

The Pinnacle Tower – Geotechnical Challenges (continued)

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Case History

Geotechnical structure damages during the 2009 Typhoon Morakot

Dr. Wei F. Lee (National Taiwan University of Science and Technology)
Prof. Ikuo Towhata (University of Tokyo)

1. INTRODUCTION

This news reports the geotechnical disasters in Taiwan in August, 2009, that were caused by the extremely heavy rainfalls associated with Typhoon Morakot. Figure 1 shows the path and date of the travel of the typhoon. It is seen herein that the typhoon remained in the Taiwan's neighborhood from August 7th to 10th. This situation resulted in an extremely heavy accumulation of rainfall. As observed by radar in Fig. 2, the wind brought a lot of cloud and water from the Taiwan Strait towards mountains in the southern part of Taiwan Island. Fig. 3 presents rainfall data during the typhoon period. From August 7th to 10th, the total rainfall in the southern part of Taiwan exceeded 1,500 mm. Furthermore, the 24-hour rainfall on August 8th was more than 700 mm in the same area. A typical rainfall data in the mountain region is the one in Fig. 4, in which the accumulated rainfall reached as much as 2,700 mm in four days.

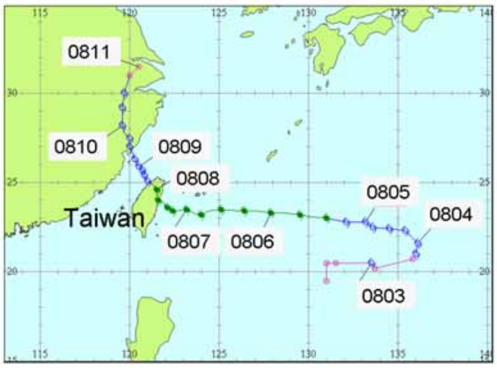


Figure 1. Path of Typhoon Morakot

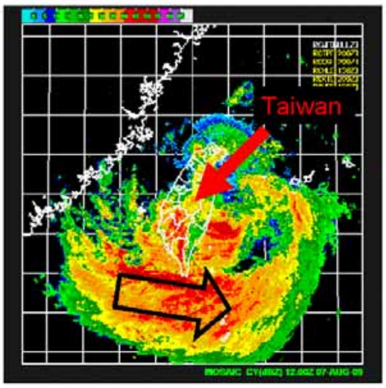


Figure 2. Vortex of typhoon cloud (radar observation)

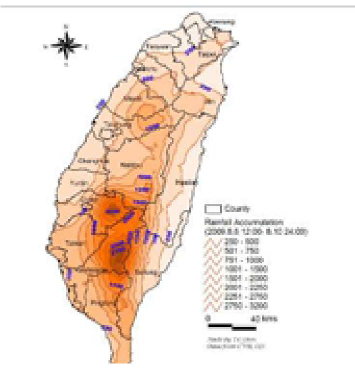


Figure 3. Data of precipitation during typhoon time(Tien-Chien Chen)

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

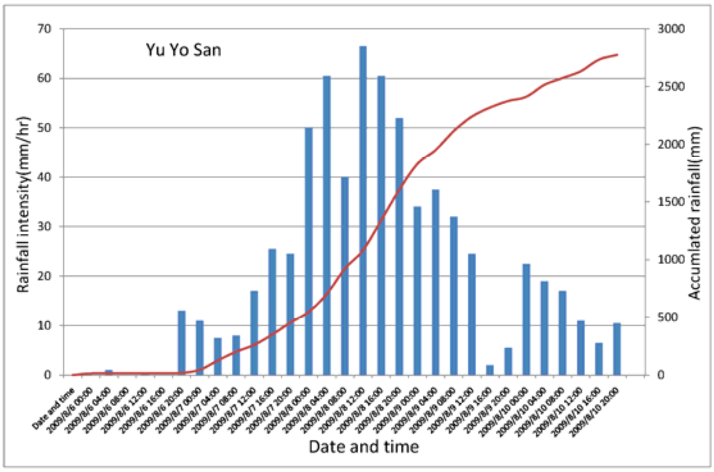


Figure 4. Rainfall data at Yu Yo San station

2. GENERAL IDEA ABOUT GEOTECHNICAL DAMAGES

The heavy rainfall in the southern part of Taiwan caused severe geotechnical damages at many places. Damage investigations revealed that they are classified into three categories;
(1) Failure of slopes : cut slope failures along roads and huge landslides,
(2) Failure of embankments such as river levees and road embankments, and
(3) Failure of bridge structures caused by scouring in foundations, collision of debris flows, and erosion of abutments.
Most of them were concentrated in the mountainous region (Fig. 5). More details of the damages are introduced in the following chapters.

3. CHEN-YU-LAN RIVER REGION

Typhoon Morakot caused a heavy rainfall in a very short time. Therefore, the river water level rose suddenly and made emergency action very difficult. Fig. 6 shows a significantly eroded river channel that endangered dwellings of people. Steep mountain slopes and valleys with deposits of weathered materials produced debris flows (Fig. 7).

Cut slopes along roads were destroyed at many places. Fig. 8 demonstrates an example in which a gravelly deposit, which was probably produced by an ancient debris flow, failed and a road structure was affected. A new road is being reconstructed in the steep slope. Noteworthy is that many retaining structures were destroyed as well. In Fig. 9, the damaged wall did not have a stable foundation and was easily affected by scouring in the base, leading to its total collapse. In Fig. 10, a ground anchor that had supported a retaining wall was pulled out.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

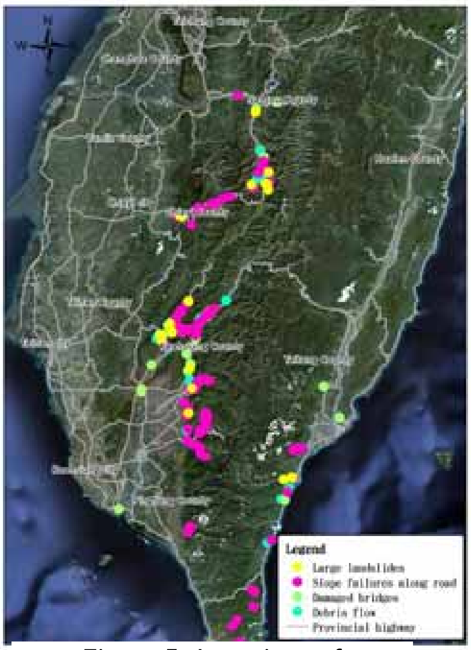


Figure 5. Locations of geotechnical damage

Figure 6. Erosion along river channel

Figure 7. Source valley of debris flow

Figure 8. Failure of natural slope along road

Figure 9. Collapse of retaining wall without stable foundation

Figure 10. Destroyed shape of ground anchor.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

Consequently, the observed geotechnical damages in the Chen-Yu-Lan river region can be summarized as what follows. First, there are many valleys that are prone to debris flow. The rapid rise of water level resulted in significant erosion along river channels. These two situations resulted in flow of huge amount of debris in the downstream areas. Second, retaining walls along mountain roads were damaged substantially. Accordingly, reconstruction of new roads is a difficult task, including construction of stable retaining structures, anti-scouring structures, and even re-routing.

4. ALI MOUNTAIN AREA

Ali Mountain is located near the central part of the Taiwan Island. Because of the steep mountain slopes and rainy climate together with its young geology, this area is substantially vulnerable to slope disasters. After the 1999 ChiChi earthquake, the loosened slopes have been producing debris in many valleys and this seems to be one of the reasons for many occurrence of debris flow in the present rainfall.

Figure 11 demonstrates an areal view of a slope failure and flow of debris. As indicated in Fig. 12, not only the surface deposits but also base rock failed. Fig. 13 shows a damaged situation of a road along a valley. It appears that the stability of this slope was affected to some extent by the erosion at its toe during the flooding. Reconstruction of road in this section poses a question about stability of this road during a future heavy rain storm.

Figure 11. Aerial view of large slope failure

Figure 12. Deep-seated failure of slope

Figure 13. Mountain road destroyed by slope failure

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

It is noteworthy that the present natural disaster was so powerful as to affect massive structures that had been considered stable under an extreme natural actions. Fig. 14 shows a rock-shed tunnel which was situated in an unstable slope. Apparently, a big amount of soils and debris came down from this slope. Fig. 15 indicates a distortion of this tunnel. It seems that this problem was induced not by the earth pressure of the flowing soil mass but the instability and deformation in the foundation of this tunnel (Fig. 16).

The lessons learnt from the damage in the Ali Mountain area are as what follows. First, the slopes in this area have been unstable and are subjected to large-scale failures. The combined effects of the present rainfall and a past earthquake shaking need to be studied further. Second, the effects of slope instability affected even such heavy structures as a rock-shed tunnel. Third, although reconstruction efforts are desperately going on, there is a fear about the stability of those slopes during future heavy rainfalls.

Figure 14. Rock shed tunnel subjected to slope failure

Figure 15. Distortion of rock-shed tunnel

Figure 16. Subsidence in foundation of rock-shed tunnel

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

5. CHI-SAN RIVER AREA

Along the Chi-San River, erosion and scouring were significant. Fig. 17 shows erosion in the toe of a cliff that affected road in the higher elevation. The power of erosion and scouring was substantial during the present event. Fig. 18 indicates a bridge that was washed away due to foundation scouring. Further, Fig. 19 indicates erosion of abutment and embankment approaching a bridge. Erosion at the toe of road embankment occurred at many places (Fig. 20).

Figure 17. Toe erosion in river channel

Figure 18. Destroyed bridge because of scouring

Figure 19. Bridge abutment and embankment destroyed by erosion

Figure 20. Erosion at toe of road embankment

6. GIGANTIC SLOPE FAILURE IN SHAO-LIN VILLAGE

One of the most tragedic events during the present typhoon was the total devastation of the Shao-Lin Village. In Fig. 21, minor streams behind the village had been considered vulnerable to possible occurrence of debris flow. During the present typhoon, a huge slope failure of 1.9×10^7 m^3 in volume occurred (Fig. 22) and formed a landslide dam in the river channel on the upstream side of the village (Fig. 23). This dam breached shortly and totally destroyed the village (Fig. 24). The number of victims reached 474.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

Figure 21. Areal view of Shao Lin Village before rainfall

Figure 22. Gigantic slope failure behind Shao Lin Village

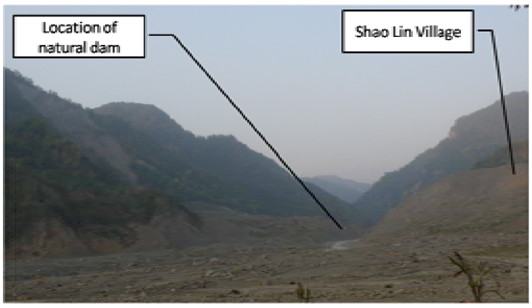


Figure 23. Location of natural dam

Figure 24. Areal view of Shao Lin Village after the disaster

7. LAO NON RIVER AREA

Many slope problems occurred in this area as well (Fig. 25). Fig. 26 indicates erosion along a river channel and consequent failure of a retaining wall. It is important to note that erosion occurred at special points alone in a curved narrow river channel (Fig. 27) probably because the direction of rapid water flow was winding and hit limited places. The same situation may occur in a bridge as well. For example, the bridge in Fig. 28 was damaged only in its part where the power of the flooding was most significant during the typhoon.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

Figure 25. Big slope failure

Figure 26. Erosion at bottom of retaining structure

Figure 27. Erosion along curved river channel.

Figure 28. Destroyed bridge

8. TAITUNG AREA

This area is situated in a coastal plane in the south-eastern part of Taiwan. Rivers brought a huge amount of debris together with water and caused damages. Fig. 29 reveals erosion of a river wall and collapse of a building as its consequence. Note that the upper photograph in this figure was taken after the water-front wall had been eroded. Thus, the scale of the flooding was greater than the design level, and the backfill soil was easily eroded.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

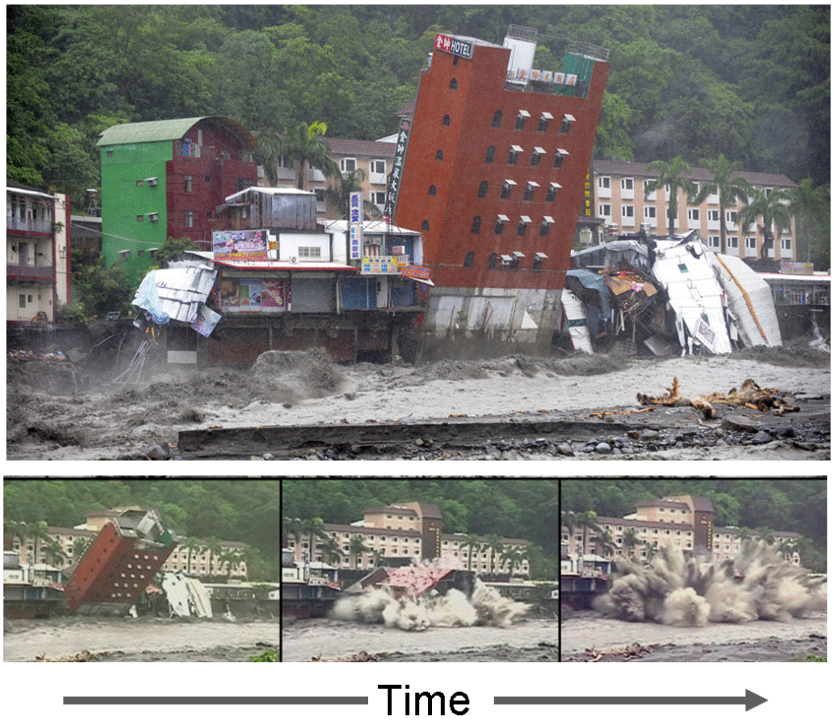


Figure 29. Erosion-induced failure of river wall and collapse of building

Figure 30. Air photograph of Taimali River channel before flooding

Figure 31. Damages in bridges and river levees

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

Figure 30 shows an air photograph of the previous topography in the mouth area of Taimali River. Although the ancient river flowed on the left (north) side, human activities shifted it towards the right bank. When the flooding occurred in 2009, the river flow destroyed the levees and came back to its original channel on the left side (Fig. 31). Consequently, both railway and road bridges were destroyed on that side. See Figs. 32 and 33 for those bridges during flooding and reconstruction.

Figure 32. Bridges during flooding

Figure 33. Reconstruction of railway bridge

9. ADDITIONAL REMARKS

The geotechnical damages caused by the Typhoon Morakot clearly indicate the problem of such an extreme natural event for which the design consideration is insufficient. The heavy rainfall triggered many slope failures in the mountain area, resulting in high water level in the downstream area (Fig. 34) and debris flow. Consequently, erosion and scouring destroyed river walls and bridges. The extent of scouring during a single flooding was unexpectedly substantial (Fig. 35) and needs more elaborate design consideration in future. Moreover, the effect of drifting woods on failure process of bridges deserves attention. As shown in Fig. 36, a huge number of trees fell down from mountain slopes into rivers, and hit bridges in the downstream areas.

It is frequently claimed from the viewpoint of global climatic change that the possibility of extreme weather condition is going to increase from now on. It is supposed that the probability of heavy rainfall may increase in the coming decades. If those ideas are meaningful to any extent, it is important to learn from the Typhoon Morakot events about the consequence and induced hazards of an extraordinary magnitude of rainfall. Erosion and scouring seem to deserve further attention.

Case History

Geotechnical structure damages during the 2009 Typhoon Morakot (continued)

Figure 34. Extremely high level of river water

Figure 35. Scouring in bridge foundation

Figure 36. Accumulation of drifting wood on bridge deck

10. ACKNOWLEDGMENT

The authors express their sincere gratitude to Taiwan Geotechnical Society who conducted very energetic reconnaissance activities after the disaster. It is hoped that the lessons from this tragedy will be useful for the safety of people in future.

Activity of Member

Hungarian National Committee

Hungarian National Committee of the International Society of Soil Mechanics and Geotechnical Engineering is pleased to announce the election on 20th of January 2010 of the President, Prof. József Mecsi, and Secretary, Dr. András Mahler. The Secretary of the Hungarian Society is based at Budapest University of Technology and Economics, Department of Geotechnics and the e-mail contact address is: contact@issmge-hungary.net.

- **XVIth Károly Széchy Memorial Session and XIXth Geotechnical Evening Forum**

The Hungarian National Committee of the Internal Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) jointly with the Geotechnical Section of the Hungarian Chamber of Engineers (MMK) organized the XVIth Károly Széchy Memorial Session on 12th February, 2010 at the Great Hall of the Hungarian Academy of Sciences, with over 250 persons attending the event.

This series of festive gatherings has been highlighted from the beginnings by lectures delivered by the most illustrious professors paying tribute to the memory of the Hungarian professor, Károly Széchy. This year, the guest speaker from abroad was Professor William Van Impe (Ghent, Belgium), president of ISSMGE from 2001 to 2005, and currently president of the Internal Association of Geoengineering Societies (FIGS).

The home speaker was Dr. Erno Biczók, invited professor at the Budapest University of Technology, formerly head of the Geotechnical Laboratory at the same University, and later technical director of the GTU Engineering Bureau in Hannover, Germany.

Traditionally, a young engineer who has excelled as the best junior speaker at the annual national geotechnical conference is offered the opportunity to introduce himself by a lecture at the memorial session. This year the candidate was Zoltán Káposztás from Geoplan Ltd.

Activity of Member

The topics of the lectures were:
William Van Impe (Belgium, president of FIGS): Analysis of observations and experience on underwater dam construction on soft subsoil.
Erno Biczók (GTU Hannover): Reinforcement and heightening of coastal flood control dams in Northern Germany.
Zoltán Káposztás (Geoplan Ltd.): An evaluation of numerous static pile loading tests from recent motorway construction in Hungary.

The two main lectures addressed the intriguing topic of underwater construction of dams and reinforcement of flood defence dams. This topic is of particularly high importance in Hungary. This country has very special hydrogeological conditions due to its being situated in the Carpathian basin, which is one of the most confined basins of the Earth, with inadequate drainage and thus exposure to ever present flood hazards hydraulic.

Following the traditions, the Károly Széchy memorial plaque and prize was awarded, this year to Dr Béla Balázsy. Professor Dr Márta Doležalová, director of Dolexpert-Geotechnika, Pague, was awarded the honorary Károly Széchy prize.

In the photo from left to right:
Dr. József Mecsi, president of Geotechnical Section of MMK, president of National Committee of ISSMGE, Dr. Béla Balázsy, Károly Széchy prize winner, Dr. Márta Doležalová, decorated with honorary Károly Széchy prize, Mrs Barsi Etelka Pataky, president of MMK, István Lazányi, honorary president of Geotechnical Section of MMK.

The professional events were concluded by an informal dinner with more than 200 participants having a good time together in very friendly atmosphere. Joyful spirits of the evening was enhanced by the toasts and the amusing speeches given by the recipients of the awards and the main speakers.

Activity of Member




Széchy Memorial Lecturers in the past:
1994 Fazakas György (Budapest), Mistéth Endre (Budapest),
Varga László (Gyor), Heinz Brandl (Wien), Farkas József (Budapest)
1996 Kovári Kálmán (Zürich)
1997 Varga László (Gyor), Lazányi István (Budapest)
1998 Heinz Duddeck (Braunschweig), Greschik Gyula (Budapest)
1999 Ulrich Smoltczyk (Stuttgart), Scharle Péter (Budapest)
2000 Dulácska Endre (Budapest), Marta Doležalová (Praha)
2001 Robert Mair (Cambridge), Müller Miklós (Budapest)
2002 Michele Jamiolkowski (Torino), Nagy János (Budapest)
2003 Jubilee memorial lecture (BME-MTA) James K. Mitchell (Blacksburg, VA)
Mecsi József (Budapest), Posgay György (Budapest), Träger Herbert (Budapest)
2004 Suzanne Lacasse (Oslo), Szepesházi Róbert (Gyor)
2005 Lothar Martak (Wien), Szabó Imre (Miskolc)
2006 Seco e Pinto (Lisboa), Szilvágyi Imre és Szilvágyi László (Budapest)
2007 Serge Varaksin (Párizs), Klados Gusztáv (Budapest and Kuala Lumpur)
2008 Roger Frank (Párizs), Soós Gábor (Budapest)
2009 Rudolf Katzenback Juhász József (Miskolc)

For a history of the Széchy Memorial Lectures, and a biography of Professor Széchy, please see http://www.issmge-hungary.net

Reported by . J. Mecsi (ISSMGE HNC)
President of the HNC ISSMGE

News

Professor K. Ishihara receives the Order of Sacred Treasure

On November 3rd, 2009, Prof. Kenji Ishihara, one of the former ISSMGE Presidents, had the Order of Sacred Treasure conferred upon him by the Emperor of Japan.

This distinction rewards Professor Ishihara's long and continuous contributions to the development of geotechnical engineering all over the world.

The International Geotechnical community is delighted to share this news.

Order of Sacred Treasure

Prof. and Mrs. Ishihara during the celebration party

Announcement

From the President of ISSMGE

The President of ISSME, Prof. Jean-Louis Briaud would like to notify the ISSMGE members that the 180 day progress report will be presented in the following issues of the Bulletin. However, short progress reports are completed every month and Prof. Jean-Louis Briaud wishes to inform all the members that these reports are fully available and can be found on the ISSMGE web site at:

http://www.issmge.org/web/page.aspx?pageid=116775

IGS News

The Symposium New Techniques for Design and Construction in Soft Clays will be held from 22 to 23 May 2010, in the city of Guaruja (about 90 km from São Paulo), Brazil, just before the 9th ICG.

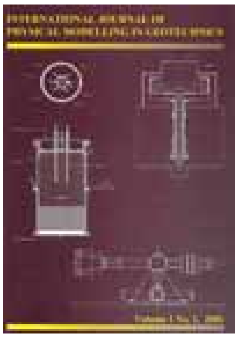
The event is organized by Brazilian Society of Soil Mechanics and Geotechnical Engineering (ABMS) and Brazilian Association of Geosynthetics (IGS Brasil) under auspices of International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) and International Geosynthetics Society (IGS).

More than 200 participants are expected in the event. Twenty eight speakers from thirteen countries will present their works in four sessions. The symposium will bring together researchers, designers, consultants, engineers, contractors, teachers and students aiming to present and to discuss the most recent developments, improvements and new technologies for design and construction in soft clays including geosynthetics.

More information may be found in http://www.geotec.coppe.ufrj.br/ssc2009/. Registration is available at the website.

Announcement

International Journal of Physical Modelling in Geotechnics



The Institution of Civil Engineers (ICE), UK is delighted to announce that it will publish the International Journal of Physical Modelling in Geotechnics from 2010. The journal will be edited by Professor David Muir Wood.

The International Journal of Physical Modelling in Geotechnics aims to cover all areas of physical modelling, at any scale, including modelling at single gravity and at multiple gravities on a centrifuge, shaking table and pressure chamber testing and geoenvironmental experiments, but excluding full scale field projects unless they are part of a programme of modelling which includes tests at a smaller scale. Papers on particular instrumentation, apparatus or procedures developed for model testing and simulation of construction processes at model scale, and papers concerned with the scaling criteria for interpretation of results of model tests for application at larger scales will be welcome. The editors will be happy to advise potential authors on the acceptability of their papers for the journal.

For information on submission of papers please see:

http://www.editorialmanager.com/ijpmg/

The International Journal of Physical Modelling in Geotechnics (IJPMG) will be hosted on the ICE's innovative ICE Virtual Library. Through the ICE Virtual Library, ICE is demonstrating its commitment to disseminating authoritative research and best-practice throughout the industry and academia. ICE will provide its full support to ISSMGE TC2 in its aim to encourage dissemination of applications of physical modelling to practising engineers and researchers. The ICE Virtual Library makes available the journal's archives since 2001: see

http://www.icevirtuallibrary.com/content/serial/ijpmg

Obituary

Dr. Leonardo Zeevaert Wiechers
His life and achievements

The international geotechnical engineering community is deeply saddened that Dr Leonard Zeevaert passed away in Mexico City on February 16th, 2010. He was born in Veracruz, Mexico, on November 27th, 1914.

He obtained a Civil Engineer degree from the National University of Mexico in 1939, a Master degree in Civil Engineering at Massachusetts Institute of Technology in 1940 and was bestowed the title of Doctor in Philosophy (Ph. D.) at University of Illinois in 1949, where he worked with Dr. Karl Terzaghi in different soil mechanics assignments.

Prof. Zeevaert at his office, ca. 2000

Professional practice. As a consulting engineer, he carried out Soil Mechanics surveys and performed analysis and design of structures and foundations for as many as 692 projects, during more than 50 years. He developed several foundation systems for highly compressible soils such as those encountered in Mexico City. He brought forward the basic theory of compensated foundations combined with friction piles and proposed a new method to estimate negative skin friction on point bearing piles.

One of the most important projects in which he had a leading participation was the *Latinoamericana Tower*, a 43-stories high building for which he performed Soil Mechanics studies, designing the foundation and acting as consulting engineer in the design of the steel structure, where the concept of controlled flexibility was applied for the first time (1947-1948). He developed a new procedure for the construction of buildings, eliminating columns in the facade to provide more architectural flexibility in the ground floor of such constructions. These ideas were introduced for the first time at international level in the design of the headquarters of "*Compañía de Seguros Monterrey*" (1960) and "*Celanese Mexicana, S.A.*", both built in Mexico City.

He was also active in the field of Coastal Engineering studying wave action on the coastline and hydraulics of marginal lagoons. He designed harbors and marinas for small boats for various sites in the Mexican Republic.

He performed the analysis and design of foundations for turbogenerators at several industrial plants and provided advice for the foundation design of an atomic energy plant in San Jose, California, U.S.A.

Research. One of the most important facets of Dr. Zeevaert work corresponds to research carried out with the purpose of developing the most appropriate analysis methods for different particular foundation systems and to forecast the seismic behavior of foundations and structures. He developed innovative methods to assess interaction between soil and structure that have become classical and are still used world wide.

From 1954 onwards he devoted time to study the problems of earthquakes and their effects on foundations and, for this purpose, he designed the "*free vibrating torsion pendulum*" with which it is possible to determine the dynamical properties of the soil. In addition, he recorded for the first time in the history of Mexico City the earthquakes of May 11 and 19, 1962, from which the response spectra of the subsoil in the downtown area of the city could be defined; these data have been used in the preparation of the building code for seismic design in the Federal District.

He developed a method to find out the resonance periods of the subsoil to be introduced in the design of tall buildings subjected to seismic forces.

He also performed important research aimed at solving problems in Coastal Engineering and in dewatering systems, as well as on the design of marinas in the Pacific Ocean.

Academic duties and publications. He was the first professor of Soil Mechanics and Foundation Engineering at School of Engineering of the National University of Mexico (UNAM) where he taught from 1941 to 2000. He was elected Emeritus Professor in 1986.

He wrote more than 200 papers on different topics of Soil Mechanics, Foundation Engineering and Earthquake Engineering. He is author of the books: "*Foundation Engineering for Difficult Subsoil Conditions*", edited by Van Nostrand-Reinhold, "*Interacción Suelo-Estructura de Cimentaciones Superficiales y Profundas Sujetas a Cargas Estáticas y Sísmicas*", from Editorial LIMUSA, and "*Seismo-Geodynamics of the Ground Surface*", a private publication.

Honors. Dr. Zeevaert was invited to deliver lectures and courses on Soil Mechanics and Earthquake Engineering in several universities of U.S.A., Europe, Central America, West Indies, South America, Democratic Republic of China, and People's Republic of China and was also invited, in 1964, to supply information on the advances of Civil Engineering which was buried in the *Time Capsule*, during the World Fair of New York. He was appointed as official delegate to a number of international conferences and presented contributions in different forums.

The American Institute for Steel Construction honored him with a special prize for the good behavior of the Latinoamericana Tower during the strong earthquake of 1957 in Mexico City. This prize was the first one awarded to the tallest building outside the U.S.A. subjected to a strong earthquake and founded upon difficult subsoil.

He was a member of the following societies: Asociación de Ingenieros y Arquitectos de México; Colegio de Ingenieros Civiles de México; American Concrete Institute; American Society of Civil Engineers; The Geological Society of America; The Seismological Society of America; Earthquake Engineering Research Institute; and Sociedad Mexicana de Mecánica de Suelos (now Sociedad Mexicana de Ingeniería Geotécnica) from which he was a founding member and President since its establishment in 1954 till 1968. He was Vice-President for North America of the International Society for Soil Mechanics and Foundation Engineering during the period 1961-1965.

In 1965 he was honored by the American Institute of Architects who bestowed upon him the gold medal "Allied Professions Medal". He also received many other professional honors, at both international and local levels which are too many (85) to mention; among them he was an honorary member of the Belgium Royal Academy of Arts and Sciences and of the National Academy of Engineering of the United States of America.

In October 27, 1987, the American Society of Civil Engineers honored him with an invitation to deliver the "*Terzaghi Lecture*" during the Convention held in Anaheim, CA., U.S.A.

Letter from the President of ISSMGE

I am very sorry to hear the sad news about Professor Zeevaert's passing. There is no doubt in my mind that he was one of the giants of our profession. I recall attending his Terzaghi Lecture in 1987 as a young professor where he talked about the Mexico City earthquake. I really enjoyed it and learned quite a bit. The clarity of the lessons learned he shared with the audience was impressive. Like Peck he was of the generation that did not need many slides but enjoyed telling stories and kept his audience captivated. I also cherish the few moments I spent with him when Walter Paniagua (President of the Mexican National Society) invited me in the early 90s to give a short course on in situ testing and foundation design in Mexico City. We really are hitting a bad period where we are losing a number of high profile people in geotechnical engineering: Peck, De Mello, Reese, and now Zeevaert.

The 18000 plus members of ISSMGE mourn the loss of Professor Zeevaert and we extend our sincere condolences and deep sympathy to his family and his close friends across the world in general and in the Mexican National Society in particular. While he is no longer with us, his work will continue to help many generations of young engineers.

Jean-Louis Briaud
February 2010

Event Diary

ISSMGE SPONSORED EVENTS

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

2010

2nd International Symposium on CPT, CPT'10
Date: 9 - 11 May 2010
Location: Hyatt Hotel & Resort , Huntington Beach, California, United States
Language: English
Organizer: TC 16 ISSMGE
• Contact person: Dr Peter Robertson
• Address: 2726 Walnut Avenue
90755 Signal Hill
California
United States
• Phone: 562-427-6899
• E-mail: probertson@greggdrilling.com
Website: www.cpt10.com

17th Southeast Asian Geotechnical Conference
Date: 10 - 13 May 2010
Location: Taipei Int'l Convention Center , Taipei, Taiwan
Language: English
Organizer: SEAGS, TGS
• Contact person: Ms. Shaan Hsieh
• Address: 4Fl., No.158, Jingye 1st Rd.
10462 Taipei
Taiwan
• Phone: 886 (2) 8502-7087 ext. 13
• Fax: 886 (2) 8502-7025
• E-mail: lisa@elitepco.com.tw
Website: www.17seagc.tw

9th International Conference on Geosynthetics
Date: 23 - 27 May 2010
Location: Sofitel Jequitimar Hotel , Guaruja, Brazil
Organizer: IGS, ABMS
• Contact person: Secretaria do Congresso - (ICG - Brazil 2010)
• Address: Av. Brigadeiro Faria Lima, 1478 sala 314,
São Paulo, SP
Brazil CEP 01451-001
• Phone: 55 11 3032-3399
• Fax: 55 11 3819-6311
• E-mail: info@9icg-brazil2010.info
Website: www.9icg-brazil2010.info

DECGE - 14th Danube-European Conference on Geotechnical Engineering (2-4 June)
Event organised under the auspices of ISSMGE
Date: 2 - 4 June 2010
Location: University of Technology , Bratislava, Slovakia
Language: English
Organizer: Slovak group of ISSMGE
Secretary: • Contact person: GUARANT International spol. s r. o.
• Address: Uhrova 10
831 01 Bratislava
Slovak Republic
• Phone: 421 2 54 430 206
• Fax: 421 2 54 430 206
• E-mail: decge2010@guarant.cz
Website: www.decge2010.sk

NUMGE2010
Date: 2 - 4 June 2010
Location: Trondheim, Norway
Language: English
Organizer: NTNU Trondheim
• Contact person: Mrs. Astrid Bye
• Address: NTNU Videre, Paviljong A, Dragvoll
7491 Trondheim
Norway
• Phone: 47 73 59 52 54
• E-mail: numge2010@videre.ntnu.no
Website: www.ntnu.no/numge2010

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/

Event Diary (continued)

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 40, 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

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 N° 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
Central Java
Indonesia
• Phone: 62-24-7622790
• Fax: 62-24 7622785
• E-mail: hpjijateng@yahoo.co.id; geoconfina@yahoo.com
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

Event Diary (continued)

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

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 ITA-AITES 2010 World Tunnel Congress and 36th General Assembly
Date: 14 - 20 May 2010
Location: Vancouver Convention Centre , Vancouver, British Columbia, Canada
Language: English
Organizer: TAC and NRC for ITA-AITES
• Contact person: Marie Lanouette, Congress Manager
• Address: National Research Council Canada
K1A 0R6 Ottawa
Ontario
Canada
• Phone: 613-993-0414
• Fax: 613-993-7250
• E-mail: wtc2010@nrc-cnrc.gc.ca
Website: www.wtc2010.org

SSC2010 - New Techniques for Design and Construction in Soft Clays
Date: 22 - 23 May 2010
Location: Sofitel Jequitimar Guarujá , Guarujá, São Paulo, Brazil
Language: English
Organizer: COPPE/UFRJ - ABMS - IGS Brasil
• Contact person: Mário Vicente Riccio Filho
• Address: Laboratório de Geotecnia - Bloco Anexo Centro de Tecnologia
Cidade Universitária UFRJ
21945-970 Rio de Janeiro
Rio de Janeiro
Brazil
• Phone: 55 21
• Fax: 55 21
• E-mail: ssc2010@coc.ufrj.br
Website: www.geotecnia.coppe.ufrj.br/ssc2010/

Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
Date: 24 - 29 May 2010
Location: Marriott Mission Valley , San Diego, California, United States
• Contact person: Lindsay Bagnall
• Address: Distance and Continuing Education, Missouri University of Science and Technology
• Phone: 573-341-4442
• Fax: 573-341-4992
• E-mail: geoeqconf2010@mst.edu
Website: 5geoeqconf2010.mst.edu

11th International Conference: Geotechnical Challenges In Urban Regeneration
Date: 26 - 28 May 2010
Location: ExCel Center , East London, United Kingdom
Language: English
Organizer: DFI and EFFC

Event Diary (continued)

• Contact person: Debbie Young
• Address: Greater London House, Hampstead Road
NW1 7EJ London
UK
• Phone: 44 (0)207 728 3910
• E-mail: debbie.young@emap.com
Website: www.geotechnicalconference.com

Geotechnical Challenges in Megacities
Date: 7 - 10 June 2010
Location: Moscow, Russia
Language: English, Russian
Organizer: NIIOSP & GRF
• Contact person: Mikhail Kholmyansky - Secretary General
• Address: 2-nd Institutskaya St., 6, build.12 NIIOSP
109428 Moscow
Russia
• Phone: 7 499 170 2709, 7 499 170 2767
• Fax: 7 499 170 2767
• E-mail: info@GeoMos2010.ru
Website: www.GeoMos2010.ru/

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 Argentinean 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, Le Belvédère 1002
Tunis, Tunisia
• Phone: (216) 22 14 66 34

Event Diary (continued)

• Fax: (216) 71 87 14 76
• E-mail: imensaid2@gmail.com, essaieb.hamdi@enit.rnu.tn
Website:
www.enit.rnu.tn/fr/manifestations/icge2010/index.html

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,
P.O.Box 16765-1719
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
PAKISTAN
• 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 Khabarovsk
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
Thailand
• Phone: 66819043060
• Fax: 6625792265
• E-mail: fengatj@ku.ac.th
Website: www.unsat.eng.ku.ac.th

2012

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

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 1 in March 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)

Acknowledgments

Mr. Makoto Namba of Brazil has recently completed his term as a member of the editing team of this bulletin. He has been a member for many years and has done perfect jobs during his time. Upon this occasion, I would like to express my deepest thanks to his contribution and efforts and I hope his coming life will be as successful as so far.

Ikuo Towhata
Chief Editor of ISSMGE Bulletin

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