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# Technical session 2c: Excavation, retaining structures and foundations

## Séances techniques 2c: Excavation, constructions de rétention et foundations

C.W.W. Ng
*Department of Civil Engineering, The Hong Kong University of Science and Technology, Hong Kong SAR*

ABSTRACT
This General Report reviews a total of thirty four papers related to excavation, retaining structures, shallow and deep foundations from sixteen countries and two regions. Thirty two papers are written in English whereas the remaining two are in French. Methodologies adopted in the papers include theoretical studies, numerical and physical modelling, field monitoring and in-situ testing. This report is intended to highlight some key features and findings and, perhaps, discuss some possible pitfalls from the papers. Readers are highly encouraged to refer to the original papers for more details.

RÉSUMÉ
Ce rapport général passe en revue un groupe choisi de trente quatre articles liés à l'excavation, aux structures de soutènement, aux foundations peu profondes et profondes. Trente deux articles sont écrits en anglais tandis que les deux restants sont en français soumis de seize pays et de deux régions. Les méthodes de recherches adoptées en ces articles incluent des études théoriques, modélisation numérique et physique, surveillance et essai en place. Ce rapport est prévu pour souligner les caractéristiques principales, les résultats principaux et, peut-être, quelques pièges possibles de chaque article. Des lecteurs sont fortement encouragés à se référer aux articles originaux pour plus de details

## 1 INTRODUCTION

A total of thirty four papers from sixteen countries and two regions are grouped in Session 2c. The themes of these papers cover excavation, retaining structures, shallow and deep foundations. A list of papers grouped under each sub-theme is given in Table 1. Among these papers, thirty two papers are written in English whereas the remaining two are in French. Research, design and investigation tools adopted in the papers include theoretical studies, numerical and physical modelling, field monitoring and in-situ testing. Due to page limit, this report only highlights some key features and findings and, perhaps, discusses some possible pitfalls from the papers. Readers are highly encouraged to refer to the original papers for more details.

## 2 EXCAVATION

*Hsi and Yu* describe an interesting and well-documented case history regarding the design and construction of a cut and cover tunnel excavated in deep marine soft clay (alluvial/fluvial deposits) up to 50m thick in Singapore. The width of the excavation ranged approximately from 40 m to 60 m and the maximum depth of excavation was 20 m. The ground water level was within 1m to 1.5m below ground surface. Due to practical and economical considerations, the retaining structure comprised of braced sheetpile walls supporting the excavation were founded in the marine clays. The total depth of the sheetpile walls was about twice the excavation depth. To provide lateral and basal stability, jet grouting slabs ranged from 3.0 m to 3.5m, were constructed. Temporary bored piles were installed to anchor the jet grout slabs and prevent uplifting of the slabs during excavations.

Two-dimensional finite element (FE) analyses using PLAXIS were carried out to assist in the design. The soil was modelled as elasto-plastic material with Mohr-Coulomb failure criterion. The interface between soil and structural element was considered and simulated by using different strength reduction factors, depending on soil and member types. Although it is not so obvious how consolidation of the soft soil was incorporated in the analysis, Class A predictions of the wall deflection are compared with field measurements as shown in Figure 1. The authors should be applauded for their willingness of presenting their Class A predictions, especially when the predictions are not exactly consistent with the field measurements. Lessons learnt from this genuine case history will no doubt benefit the geotechnical community significantly in the long run.

*Blackburn et al.* report field monitoring of steel strut loads inside a 10m deep excavation in clay in the Evanston campus of Northwestern University. The soil stratigraphy consisted of 5m of sandy urban fill overlaying a 1m clay crust, 4.3m of soft clay and 7.9m of medium clay. Strain gauges were placed at the mid-points of cross-lot bracing and diagonal support members, which supported the approximate 37m by 44m (on plan) and 10m deep excavation retained by sheetpile wall. The vibrating wire strain gauges were carefully placed to separate axial and bending stresses. It is reported that axial thermal loading may constitute 50% of the axial strut load. Thermal and self-weight induced bending stresses approximately equal to those arising from lateral earth pressure and thermally-induced axial stresses. Corner effects were relatively insignificant at this site. Similar significant temperate effects on steel struts and diaphragm wall are reported by Ng (2002) who measured daily cyclic deflections of a 0.6m thick concrete diaphragm wall as a result of daily temperature variations of about $10^{o}$. Clearly there is a need to monitor temperature effects steel struts and walls when significant temperature variations are expected.

*Placzek* describes design considerations of a 50m diameter and 60m deep circular shaft in difficult ground conditions. However, details of the ground conditions are not reported and the actual location of the shaft is not clear. In the design, external earth and water pressures are resisted by the hoop stress of the shaft. No external tieback and internal horizontal structural bracings are provided to support the shaft. By using the so-called Bedding (Subsoil Reaction) Modulus Method, three design cases including two geometrical imperfections are

Table 1. A list of reviewed and reported paper in the session of Excavation, Retaining structures and Foundations.

| Title of Paper | Author | Country |
|---|---|---|
| **Sub-theme: Excavation** | | |
| 1. Jet grout application for excavation in soft marine clay | Hsi, J.P., Yu, J.B.Y | Australia |
| 2. Observed bracing responses at the Ford Design Center excavation | Blackburn, J.T., Sylvester, K., Finno, R.J. | USA |
| 3. Load bearing capacity of large-size, circular excavation walls without horizontal supporting systems. | Placzek , D. | Germany |
| 4. Design and numerical investigations of a deep excavation for a tunnel entrance pit | Raithel, M., Gebreselassie, B., Müller, S., Pahl, F. | Germany |
| 5. Back analyses and safety prediction for an extremely deep foundation pit during its excavation | Song, E., Lou, P., Lu, X. | China |
| 6. A study on the method for design and construction management considering strain level of ground during excavation | Takahashi, K., Okochi, Y. | Japan |
| 7. Validation of design methods with in situ monitoring of deep excavations. | Korff, M., Herbschleb, J. | Netherlands |
| 8. Excavation induced building damage | Portugal, J.C., Portugal, A., Santo, A. | Portugal |
| 9. Deformations of the buildings located near foundation trenches and underground excavations and the measures for their reduction | Ilyichev, V.A., Konovalov, P.A., Niki-forova, N.S. | Russia |
| **Sub-theme: Retaining structure** | | |
| 1. Pile-Soil-Wall-Interaction during the construction process of deep excavation pits | Katzenbach,R., Bachmann, G., Gutberlet, C. | Germany |
| 2. Modelling of horizontal earth pressures on retaining walls | Chua, H.Y., Bolton, M. | UK |
| 3 Mobilization of the earth resistance of a normally consolidated cohesive soils | Gebreselassie, B., Kempfert, H.-G. | Germany |
| 4. Seawall construction in Moreton Bay, Brisbane | Ameratunga, J., Shaw, P., Beohm, W.J., Boyle, P.J. | Australia |
| 5. Observed behaviour of a quay wall at the new 'Port 2000' at Le Havre, France (in French) | Marten, S., Delattre, L., Pioline, M., Vinceslas, G., Joignant, Ph., Lavisse, J. | France |
| 6. A simplified procedure to evaluate earthquake induced displacement of gravity type retaining walls | Koseki, J. | Japan |
| **Sub-theme: Shallow Foundation** | | |
| 1. The effectiveness of buried mass concrete thrust blocks as a means of lateral support for excavations | Goodey, R.J., McNamara, A.M., Taylor, R.N. | UK |
| 2. Bearing capacity analysis of shallow foundations from CPT data | Eslami, A., Gholami, M. | Iran |
| 3. Novel centrifuge simulations of restoration of building tilt | Ng, C.W.W., Lee,C.J, Xu,G.M., Zhou, X.W. | HKSAR |
| 4. Continuum approach for analysis of short composite caisson foundation | Ali Jawaid, S.M., Madhav, M.R. | India |
| 5. The relevance of the yield shear strength of plastic clays in the bearing capacity of foundations | Graterol M. J. | Venezuela |
| 6. Rigidity characteristic and deformation calculation of large-area thick raft foundation | Gong, J.F., Huang, X.L., Teng, Y.J. | China |
| 7. Vibration isolation of foundations subjected to impact loads by open trenches using physical models | Jafarzadeh, F. | Iran |
| 8. Centrifuge modelling of soil upheave by expanding tubes | Wichman, B.G.H.M., Allersma, H.G.B. | Netherlands |
| 9. Densification of hydraulic fills by vibroflotation technique | Mecsi, J., Gokalp, A., Duzceer, R. | Hungary /Turkey |
| **Sub-theme: Deep Foundation** | | |
| 1. Foundation engineering for the UK's new national stadium at Wembley | O'Brien, A.S., Hardy,S., Farooq,I., Ellis,E.A. | UK |
| 2. The foundations of the 2nd railway bridge of Argenteuil (in French) | Bustamante, M., Bourgeois, E., Gianeselli, L., De Justo, J.-L. | France |
| 3. Foundation design for a new cable-stayed bridge crossing the Panama Canal | Moormann, Ch., Humpf, K. | Germany |
| 4. Complex foundation design in inhomogeneous ground conditions for a high-rise building in Frankfurt, Germany | Quick, H., Keiper, K., Meissner, S., Arslan, U. | Germany |
| 5. Underpinning of foundations in collapsing soils | Compagnucci, J.P. | Argentina |
| 6. Geotechnical analyses of Taipei International Financial Centre (Taipei 101) Construction Project | Lin, D.-G., Woo, S.-M. | Taiwan |
| 7. Foundation of a tall building in cavernous limestone | Reul, O., Ripper, P. | Germany |
| 8. The peculiarities of stress-strain state at interaction of high rise buildings and structures with the base | Boyko, I., Saharov, O., Nemchynov, Yu. | Ukraine |
| 9. Mechanical behaviour of caisson foundation reinforced by steel pipe sheets piles | Isobe, K., Kimura, M. | Japan |
| 10. Some problems of the founding of the powerful turbo-generator sets | Taranov, V.G., Shvets, N.S., Shvets, V.B. | Ukraine |

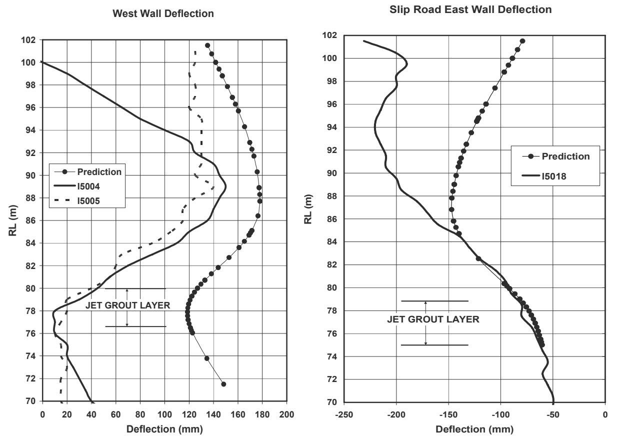


Figure 1. Measured and predicted (Class-A) sheet pile deflections (Hsi and Yu).

considered and analysed. The Observational Design Method is employed together with a monitoring programme. However, neither any contingent plan nor actual field measurement from the shaft is reported and discussed in the paper.

*Raithel et al.* introduce and discuss re-design of a 12.5 m deep excavation adjacent to a 12.5 m high cofferdam, which is located at a distance of 8 m away from the excavation (see Figure 2). The deep excavation is retained by an 800 mm thick diaphragm wall together with 4 prestressed anchors installed at 4 phases (stages). The ground is characterized by a succession of sand, basin silt and boulder clay. The ground water condition is strongly affected by the water level in Trave river in Lubeck in Germany and two aquifers.

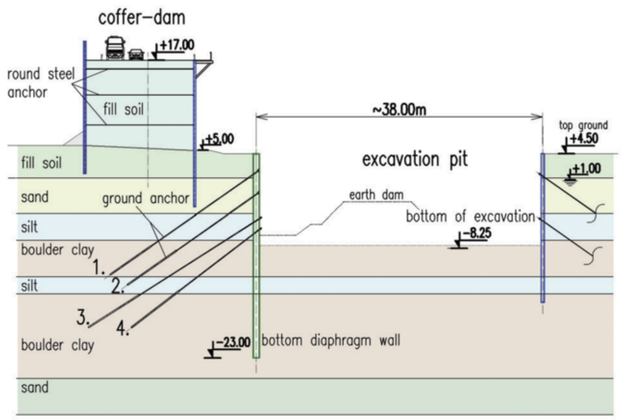


Figure 2. Cross section of the excavation after the re-design (Raithel et al.).

The finite element programme – PLAXIS was used to assist in the re-design of the staged excavation. A hardening soil model called HSM was adopted and interface elements were used. Prior to the simulation of the deep excavation, detailed stress history of the ground was modelled. Drained analysis was carried out at all phases (stages) of the deep excavation. However, it is not obvious how well of the HSM can capture the intended inclusion of the recent stress history effects in the analysis. In order to do so, any soil model used must be able to capture the stress and strain dependent characteristics of soils when a multi-stage deep excavation is simulated (Ng and Lings, 1995; Ng et al. 1998; Ng et al. 2004). Figure 3 compares the measured and computed wall deformations of the cofferdam. Accordingly to the authors, the re-design was based on the numerical analysis and the project was safely executed and completed.

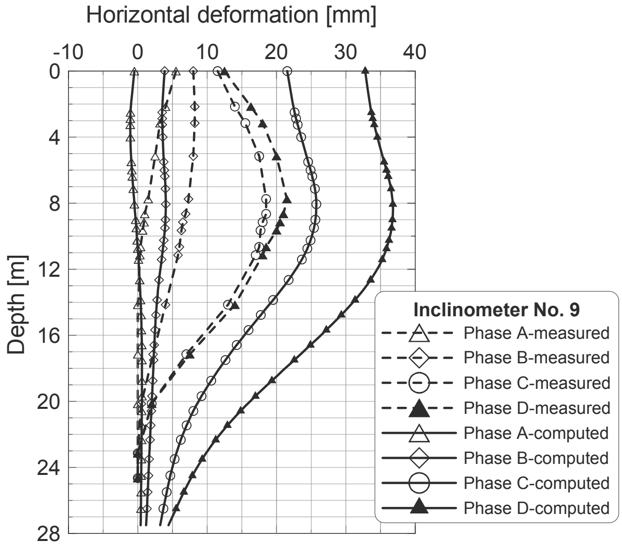


Figure 3. Measured versus computed deformations (Raithel et al.).

*Song et al.* report a 50 m deep excavation for the foundation (north) anchor block of the Runyang Yangtze Bridge in China. The north block is located on a small island in the Yangtze river. The top 20 m of soil layers consist of essentially soft silty clays which are underlain by about a 30 m thick medium to dense fine sand. The ground water table is close to the ground level. The 69m by 50m (on plan) excavation is retained by a 1.2m thick concrete diaphragm wall socketed into weathered rocks ranging from 1m to 6m, depending on the degree of weathering. Inside the excavation, 11 levels of struts were installed during the excavation.

In this project, the finite element package ANSYS was used to analyse the excavation three-dimensionally. The soil and the concrete were modeled as Drucker-Prager and bi-linear hardening materials, respectively. It is not totally clear whether effective stress or total stress analysis was carried out. Unusual material parameters for silty clays (i.e., c=20-25 kPa; $\varphi$=5-9$^o$) and for silty sands (i.e., c=10 kPa; $\varphi$=27-30$^o$) are listed in the paper. No information is given on how to derive these parameters. As reported, the authors adopted an optimization technique to assist in selecting model parameters as the excavation proceeded and measured. Figure 4 shows the comparison of the measured and predicted maximum displacements. The agreement between the two sets of displacements is really remarkable. However, it is rather difficult to understand and comprehend how this excellent agreement has been achieved with such a relatively simple soil model. What does this agreement mean in terms of advancing scientific knowledge?

*Takahashi and Okochi* describe design and back-analysis of a very complex central shaft excavation connecting to two shield tunnels to form the Rinkai Oimachi station. Two- and three-dimensional finite element analyses were carried out to assist in the design (prior analysis) and the back-analysis (post analysis). The location of ground water table is not given. They conclude that a nonlinear elasto-plastic analysis using a hardening soil model do not give better results than those from a linear elastic analysis. However, it is not obvious whether effective stress or total stress analysis was carried out. Reviewing the soil parameters used (refer to Table 2), it becomes clear that many soil parameters listed in the table do not appear to make any physical sense. For example, there is a significant inconsistency between the SPT value (N=10) and the $E_{50}$ and $E_{oed}$ values of the Tokyo clay. Moreover, the two strength parameters (i.e., C=130 kN/m$^2$ and $\phi$=17$^o$) for the clay are somewhat inconsistent with each other and the high C value is difficult to be understood. Are they effective or total stress parameters ?

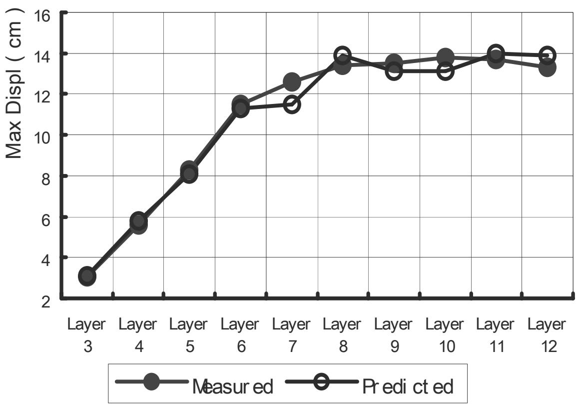


Figure 4. Comparison of the measured and predicted maximum displacements (Song et al.).

*Korff and Herbschleb* introduce and discuss results of field measurements and analyses of at two deep excavations in typical soft soil conditions in the Netherlands at the Sophia Rail Tunnel and in sands mainly at the Tunnel Pannerdensch Kanaal, respectively. They make some recommendations on field monitoring and comment on the current analysis methods for designs.

*Portugal et al.* describe the methods proposed by Burland and Wroth (1977), Burland (1997) and Rankin (1988) for assessing risk of damages to building due to underground constructions and then the authors illustrate the use of the methods with two hypothetical examples.

*Ilyichev et al.* identify key factors affecting the deformations of buildings due to adjacent underground work and excavations and derive some empirical correlations for assessing and designing protective measures. It appears that their research work is based on the study of the deformations of 73 affected buildings with the aid of the finite element programme - PLAXIS.

Table 2 Soil parameters for the hardening model (Takahashi and Okochi)

| Soil | Thickness m | N: SPT | $\gamma_t$ kN/m$^3$ | $E_{50}$ kN/m$^2$ | $E_{oed}$ kN/m$^2$ | C kN/m$^2$ | φ ° | ν |
|---|---|---|---|---|---|---|---|---|
| Loam | 10.7 | 5 | 13.57 | 50000$^a$ | 37000$^a$ | 80$^a$ | 17$^a$ | 0.48 |
| Musashino gravel | 4.1 | 50 | 19 | 240000$^b$ | 144000 | 0.1$^c$ | 42$^c$ | 0.3 |
| Tokyo clay | 17.1 | 10 | 18.65 | 100000$^a$ | 59000$^a$ | 130$^a$ | 17$^a$ | 0.48 |
| Tokyo gravel Edogawa sand | 28.1 | 50 | 18.87 | 735000$^b$ | 441000 | 0.1$^c$ | 42$^c$ | 0.3 |
| Grouted part | — | — | 18.87 | 1500000$^b$ | 900000 | 0.1$^c$ | 42$^c$ | 0.3 |

Note: $^a$Triaxial test; $^b$1/2PS logging; $^c$Empirical equation for SPT.

## 3 RETAINING STRUCTURE

*Katzenbach et al.* investigate influence of foundation piles inside an excavation on the reduction of soil heave and lateral wall deflections during construction and the increase in passive resistance of the soil wedge in front of a retaining wall or so-called dowelling effects (refer to Figure 5). Certainly, this is a very important issue to be studied and applied in designs. Based on the results of a series of 1:50 scale model tests in dry sand carried out under one Earth's gravity (1g), the authors verify their soil constitutive model, model parameters and numerical simulation procedures. It appears that they use a modified Drucker-Prager/cap model, which has two yield surfaces, i.e., a pressure dependent perfectly shear failure surface and a compression cap yield surface. The authors manage to show excellent agreements between the measured/deduced and computed earth pressure-wall displacement relationships. The agreements are somewhat surprising because the state-dependent dilatancy of soils, effects of suction and soil-wall interface are not seemingly considered. It is well-known that significant contributions from strong dilative behaviour of sand at low stress level in small scale model tests conducted at 1g should be considered carefully and taken into account. Some more advanced state-dependent soil models such as by Li and Dafalias (2000) and by Chiu and Ng (2003) may be worthwhile to be considered in future.

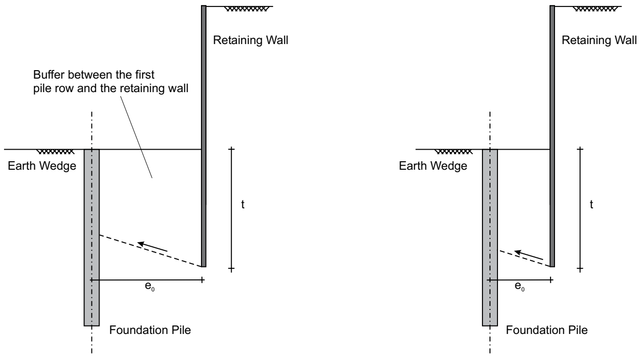


Figure 5. Section view through the earth wedge at different pile-wall-distances $e_o$ (Katzenbach et al.).

After the verification of the soil model and simulation procedures, the authors carry out a series of parametric numerical experiments to study the influence of embankment depth of the retaining wall, distance of the first row piles to the wall, pile spacing and diameter of piles on the dowelling effects attributed by foundation piles constructed in Frankfurt clay.

*Chua and Bolton* describe and report a very interesting series of centrifuge model tests at 45g to investigate the horizontal arching mechanism of a fixed cantilever wall. Figure 6 shows the plan view of a 300mm high (model scale) L-shaped model basement. The model basement comprised separate but contiguous panels of different widths and thicknesses. The flexible cantilever panels are numbered from 1 to 5, corresponding to widths of 1.8m, 3.6m, 2.7m, 2.7m and 9m at prototype scale, respectively. Thin panels were 302mm thick, medium-stiff panels were 403mm thick, and stiff panels were 604mm thick at prototype. Leighton Buzzard silica sand (Fraction E) with different densities was used in the tests. Sodium Polytungstate solution retained inside a rubber bag was used to generate stress levels corresponding to K=1 conditions inside the basement. The sand outside the model basement would have created an earth pressure coefficient $K_o \approx 1\text{-}\sin\phi'$ in the absence of wall movement, which would have generated lower stress levels. However, as demonstrated by Ng et al. (1995; 1999) and Powrie and Kantartzi (1996) in-situ walls constructed in different ground conditions do reduce lateral earth pressures significantly during construction. The horizontal arching and vertical downward shear mechanisms due to diaphragm wall installation are also identified and explained with the assistance of three-dimensional numerical analyses (Ng and Yan, 1998; 1999).

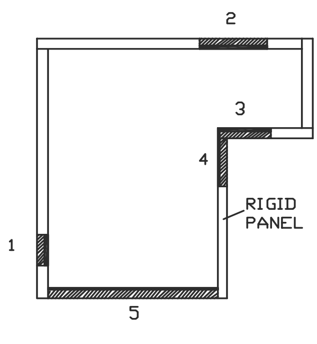


Figure 6. Plan view of model basement (Chua and Bolton).

In the current centrifuge tests, deformations of the cantilever retaining wall were induced by dropping the level of fluid in stages inside the basement. To interpret the test results, a theoretical model of a fixed cantilever using conventional Rankine's triangular pressure distribution was derived to calculate wall deflections and hence the theoretical equivalent earth pressure coefficient for any given depth of fluid level inside the basement.

Figure 7 shows the displacements of thick panels during excavation, where theoretical active values are 0.198 for a perfectly smooth wall and 0.182 for a perfectly rough wall, taking the angle of friction to be 42˚. The calculated ultimate equivalent earth pressure coefficients are smaller than that an engineer could otherwise have calculated with the same angle of internal friction. It was found that equivalent earth pressure coefficients K dropped to below active earth pressure values, even accounting for wall friction. This was found to be true for both thin and thick panels. This demonstrates that conventional design for narrow, flexible panels is over-conservative and that a horizontal arching mechanism may be employed to benefit retaining structure construction methods.

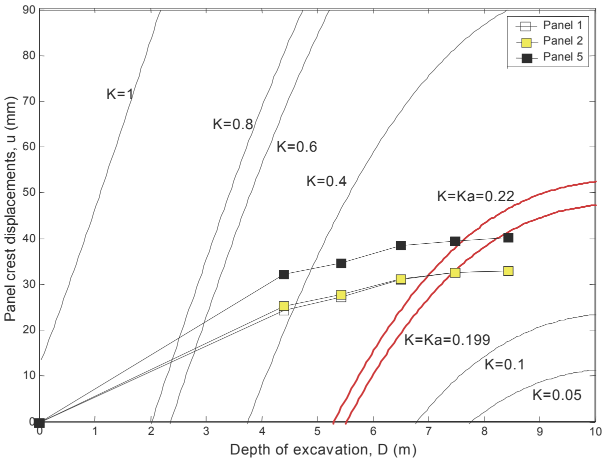


Figure 7. Superimposition of recorded thick panel displacements on predicted panel displacements (Chua and Bolton).

*Gebreselassie and Kempfert* present an attempt to develop a soil stiffness dependent earth pressure resistance function in normally consolidated soft soils by using the FEM. Three types of wall movement are considered (see Figure 8). A homogenous soft soil is assumed and simulated by a constitutive soil model known as the Hardening Soil Model (HSM), which uses the plasticity theory and appears to capture the dilatancy of soils. The HSM also considers the stress dependent stiffness of the soils according to a power law. The contact between the soil and the wall is simulated by interface elements using the Mohr-Coulomb-Model (MCM). The location of the water table is not obvious in the paper, although effective model parameters are used with saturated unit weight of the soil. Various computed results and fitted equations are provided. However, the computed results have to be verified by model tests and any other independent means.

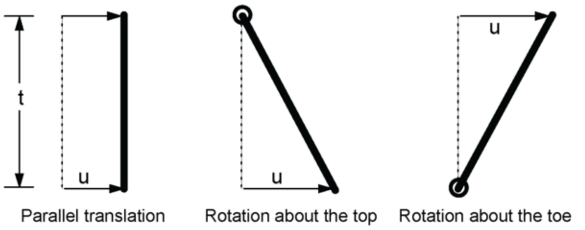


Figure 8. Types of the wall movement (Gebreselassie and Kempfert ).

*Ameratunga et al.* describe design, construction and monitoring of a 4.6km long seawall east of the existing reclaimed areas at the Port of Brisbane. The construction of the seawall is the first step in allowing the Port of Brisbane Corporation (PBC) to reclaim an additional 230ha for the future expansion of the Port. The consistency of the soft clay at the seabed surface is very soft to soft, with undrained shear strengths as low as 5 kPa. The final East Bund seawall design was a sand/rock embankment. The remaining seawall sections consisted entirely of rock. The east bund has the following key features:

1) Basal high strength geotextile (ult. capacity of 700kN/m).
2) Sand pancake wider than the basal geotextile.
3) Filtration geotextile to cover the sand and to minimise sand movements due to tides and waves.
4) Rock core bund above the sand.
5) Armour.

The authors conclude that high strength geotextiles provide a sound base when operating on very weak soils and field trials provide valuable information for the designers.

*Marten et al.* introduce the construction technique used for installing a diaphragm wall for the new deep quay "Port 2000" at Le Havre and report the instrumentation results such as displacements of the diaphragm wall and tensions in the ties. Based on the initial observed data, coupled numerical modelling was carried out to back analyze the performance of Phases 1 and 2, and to predict for the Phases 3 and 4 of the project.

*Koseki* proposes a simplified procedure to evaluate the earthquake-induced displacement of gravity type retaining walls. The proposed procedure is based on a series of comprehensive dynamic model tests at 1g. Sinusoidal shaking tests were conducted on two models with different ground conditions. One model was constructed on level ground, while the other model was constructed on a sloping ground. The gravity retaining wall was 530 mm high with a base width of 230 mm. In a sand container, a horizontal bed of dry dense Toyoura sand was backfilled and retained by the wall and subjected to 20 cycles of excitations at a frequency of 5 Hz, where the amplitude of the base acceleration was initially set to about 50 gals and increased at an increment of about 50 gals.

Figure 9 shows the formation of failure plane in the backfill for the model constructed on 5 cm-thick level ground (after shaking at 350 gals). The effect of strain softening in the backfill is considered in the evaluation of the threshold acceleration and the threshold overturning moment to be employed in the Newmark's sliding block method. Reasonable agreements are obtained between the measured results and the simulations by their proposed method. However, it is not clear how the effects of strong dilation of the dense sand at the low stress levels are considered in their analyses and predictions.

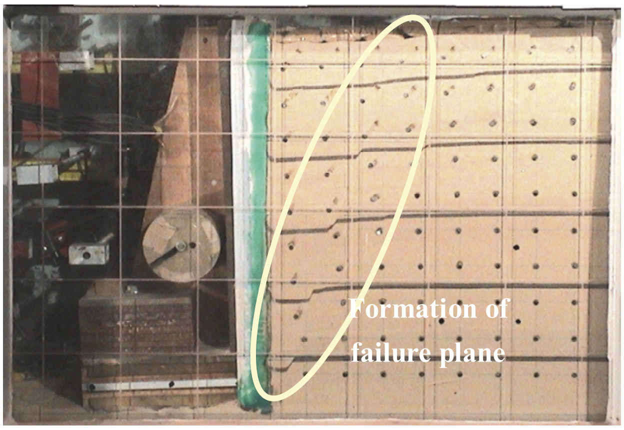


Figure 9. Formation of failure plane in the backfill after shaking at 350 gals (Koseki).

*Goodey et al.* present a very interesting and preliminary series of centrifuge model tests for investigating the resistance and displacement of model thrust blocks subjected to inclined loads in fine sand. Shallow embedded concrete blocks are frequently used to provide lateral support to raking props in temporary excavation works in the field. In the centrifuge, preliminary investigations are conducted on model thrust blocks embedded within the soil bed and subjected to equivalent prop loads (see Figure 10). A lead screw driven by an electric motor was used to apply inclined prop forces onto thrust blocks embedded near the surface. Relationships between the applied load on the block and its displacement were measured for a range of inclination angles ($20^{o}$-$40^{o}$), block sizes and embedded depths. The measured responses in the centrifuge indicate that the thrust blocks exhibit a similar general behaviour to vertically loaded foundations with an initial stiff response followed by a change in response as plastic strains develop in the soil. The foundation is also pushed to greater depth into the soil with increasing in-situ effective stress. Even though these are only preliminary tests, there is clear evidence of a different mechanism of load transfer into the soil surrounding the thrust block, depending on the inclination of the prop load.

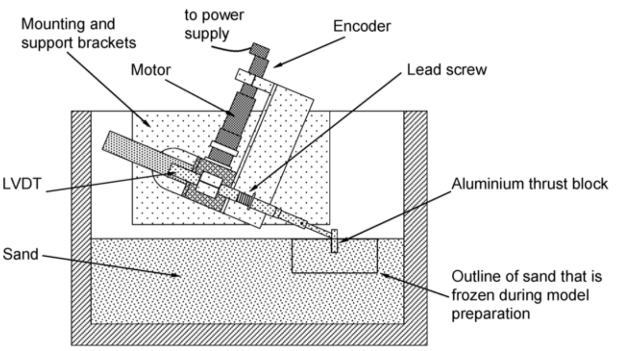


Figure 10. General set-up of apparatus (Goodey et al.).

Based on the general shear failure mechanism of a logarithm spiral, *Eslami and Gholami* introduce a new analytical model to calculate the static bearing capacity of footings, $q_{ult}$ from cone resistance $q_c$ directly. The transform of failure mechanism from shallow to deep, foundation dimension and data processing are considered in the newly proposed method. Six current CPT direct methods for determining the bearing capacity of footings are investigated. The results from the proposed method and these six methods are compared with the measured capacity of 21 full-scale tests on footings and plate load tests. Based on the statistical and cumulative probability approaches, it is reported that the average absolute error for the six current methods is 51% with an average standard deviation, SD 36%, whereas for the new method the absolute error is 17% with a SD=15%. The real challenge for this new method is to compare its Class-A predictions with measured results from different load tests in various ground conditions.

*Ng et al.* describe and report two novel centrifuge model tests to correct an initially titled building in-flight using a computer controlled 4-axis robotic manipulator (Ng et al. 2002). The objectives of the tests are to investigate the effectiveness of the soil extraction method and key factors governing the restoration of building tilts. In the centrifuge simulation tests, the robotic manipulator was used to core and extract soil close to an initially tilted model building in-flight. This soil extraction was to induce stress release, thereby mitigating the inclination of the building.

Figure 11 illustrates a sectional view of the entire model package. A large (1.5m by 1.5m by 1m height) three-dimensional model container was used to carry the 4-axis robot manipulator. Inside this large container, a smaller inner container was used to accommodate the model soil and the model building. During the centrifuge tests at 52g, the size of the simulated prototype shallow foundation was 9.36m by 9.36m on plan and the building generated an average bearing pressure of about 150 kPa, which is equivalent to a 30 story high residential building. The model soil was made of unsaturated completely decomposed granite (CDG) and prepared in three layers by compaction to a dry unit weight of 13.60 $kN/m^3$ and 16.29 $kN/m^3$ with water content of 16.6% and 13.5% in tests 1 and 2, respectively. Initial soil suction was measured prior to each test. Lateral displacements of the building were measured by laser sensors as shown in Figure 11.

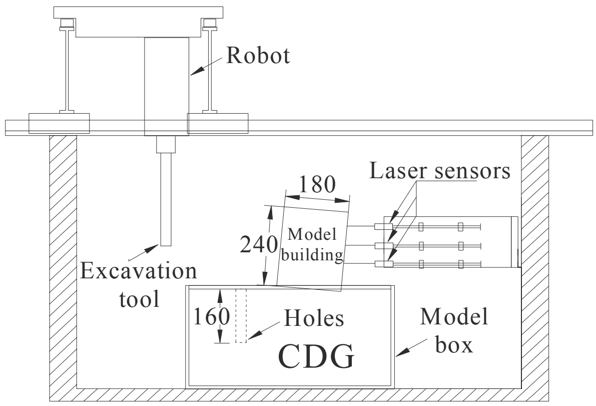


Figure 11. A sectional view of model package (unit: mm in model scale) (Ng et al.).

Nine 1.5m-diameter and 8.3m (prototype) deep holes were drilled by the robot in-flight in each test. Two different configurations of the drilled holes were studied. Figure 12 shows a reduction in building tilt with the execution of the drilling sequence in each test. At the end of drilling, the inclination of building was reduced by 0.80% and 0.48 % in test 1 and test 2, respectively. It is demonstrated that the robot can be a very effective tool for conducting parametric experiments to study of the restoration of building tilts in various ground conditions.

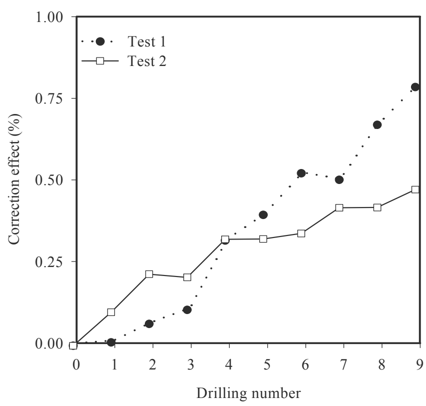


Figure 12. Reduction of building tilt (Ng et al.).

*Ali Jawaid and Madhav* analyze load – settlement behaviour of short composite caisson foundation (see Figure 13) using a continuum approach. The well steining is considered to be rigid and treated as an incompressible cylinder while the core inside is analyzed as a compressible pile. By integrating the Mindlin's solution for a vertical point load inside a semi-infinite medium numerically, soil displacements are calculated at mid points of the outer surfaces of each element, and at the centres of bases of steining and granular core. Subsequently, a detailed parametric study is carried out to evaluate the relative influence of governing parameters on the overall deformation response of the composite foundation. The parametric study quantifies the effects of length-diameter ratio ($L/d_0$), diameter ratio ($d/d_0$), modulus ratio ($E_{gp}/E_s$) and friction angle ($\phi$) on the sharing of the applied load by the well steining and the granular core and on settlement. Obviously Class-A predictions on properly conducted physical model tests and field trials are necessary to verify the calculation method developed.

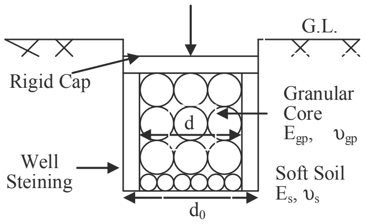


Figure 13. Short composite caisson foundation with granular core (Ali Jawaid and Madhav).

Considering saturated normally and overconsolidated plastic clays, *Graterol* carries out a comparative analysis of the bearing capacity of foundations using the undrained shear strength ($c_u$, $\phi_u = 0$) versus the use of the so-called yield shear strength, $S_c$, in the paper. The yield shear strength appears to be determined from shear tests by plotting shear stress versus displacement.

*Gong et al.* report a series of model tests with the objective to investigate and determine the combined rigidity of a large (say, 10,000 $m^2$ ) and thick raft foundation and its superstructure. Based on the model tests and field measurements, a simplified global calculation model (refer to Figure 14) is proposed to calculate the settlement of a large and thick raft foundation supporting a multi-story building.

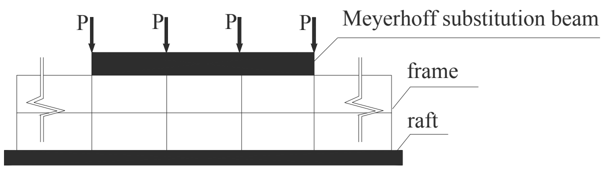


Figure 14: Simplified computation model (Gong et al.).

*Jafarzadeh* describes a series of 1g model tests to investigate the effectiveness of an isolated open trench for minimising vibration effects from machine foundations. Open trenches around an impact dynamic source was built inside a metal container with the dimensions of 100 by 100 by 80 $cm^3$. Loose dry sand was used in the tests. Dynamic impulse tests were performed by dropping a tamper which produced the required impact loads. Comparable tests were performed for identical models with and without isolating trenches around the dynamic source. It is found that with an increased depth of the trench, the efficiency of isolation is also increased. The optimum depth of the trench is affected by many parameters like the type and frequency content of the dynamic impulse loading. It appears that stress effects on the characteristics of wave propagation are not explicitly addressed in the paper.

After correctly modelling stress level in centrifuge, Itoh et al. (2002) carried out a series of novel model tests at 50g by using a new multiple ball dropping system to study and devise effective barriers to minimise wave propagation in soil. Their tests were extremely well-instrumented and excellent results were reported. Itoh et al. concluded that soft barriers were generally superior to stiff ones and the geometry of barriers could also significantly affect their performance. Soft hollow cylindrical barriers were effective when their embedded depths were more than 60% of the wavelength of the waves generated.

*Wichman and Allersma* describe and discuss a very interesting series of novel centrifuge experiments to study the mechanism and effectiveness of using expansion tubes to reduce settlements underneath road embankments. In the field, tubes of woven material are thrust horizontally into a road embankment by means of steel tubes. The steel tubes are then withdrawn while filling material is being injected to expand the woven tubes (expanders) to a diameter of approximately 800 mm. In the centrifuge, this settlement reducing technique was simulated by using in-flight expanding tubes (see Figure 15) in a dry sand model in the Delft's centrifuge.

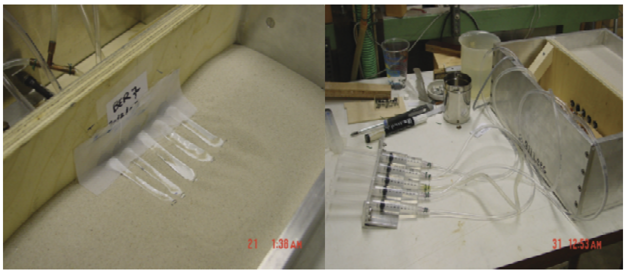

Figure 15. Test setup for simulating expanding tubes in the centrifuge: a) location empty tubes; b) syringe system for expanding tubes (Wichman and Allersma).

The authors conclude that it is feasible to lift road embankment by means of filling a horizontal array of expanders. The distance between the expanders can be three times the diameter of the expander or more, depending on its depth. Perhaps, model clay embankments may be tested at next stage of the study.

*Mecsi et al.* describe the densification of hydraulic fills by the vibroflotation method. Ideally, this paper should be grouped and discussed in Session 2a – Ground improvement. Since relatively shallow depth is involved in the densification process, the paper is now placed under Shallow Foundation loosely. According to the authors, a total of 1,034,000 $m^3$ of hydraulic fills covering an area of 181,000 $m^2$ are densified. Effects of soil densification on the hydraulic fills are investigated based on CPTU tests. Compaction energy and corresponding increase in cone resistance, $\Delta q_c$ are examined for different types of soils. Based on quality control tests, it is concluded that the average relative density of the compacted hydraulic fill successfully exceeds the specified minimum relative density of 70 %. It is also observed that the cone resistance ($q_c$) could be increased by a factor of 1 to 4. On the other hand, it is found that even thin layers of silt and clay present in hydraulic fill can negatively affect the densification process. Measured settlements are in good agreement with the calculated values by the Janbu method.

## 5 DEEP FOUNDATION

*O'Brien et al.* report a very well-written design case history concerning some geotechnical issues associated with the foundation design for the UK's new Wembley stadium located in the North London. The geology of the site is relatively simple comprising London Clay, beneath made ground of varying thickness, over the Lambeth Group and then Chalk. Measured vertical pile capacity and load-deformation behaviour of horizontally loaded piles are compared with their corresponding predictions. Moreover, the observations and predictions of settlement with depth and time below a deep area of fill are presented (see Figure 16).

*Bustamante et al.* describe and discuss re-design of a new railway bridge foundation at Argenteuil. Due to various constraints including the immediate vicinity of an old bridge built in 1864, the presence of karstic strata and the navigation on the Seine, micro-piles of very high capacity (6 MN) were chosen as the foundation for the 2nd bridge of Argenteuil. All the data related to the load test on an instrumented micro-pile, as well as the design and observations made during the grouting of the working micro-piles are reported.

Based on information obtained from the load test on the micro-pile, optimization of pile dimensions, operations of drilling and sealing were planned and carried out. Numerical modelling was also conducted to investigate an integration of group effect and contribution from pile cap.

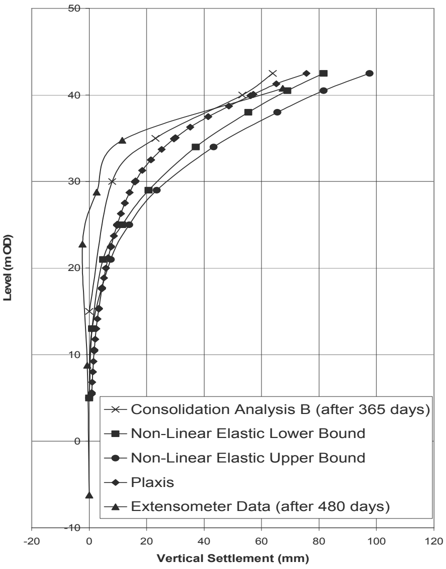


Figure 16. Predicted and observed settlement behaviour with depth (O'Brien et al.).

Table 3. Results of static pile load tests in the Cucaracha-Formation (Moormann and Humpf).

| test pile | P2 | P3 | T1 |
|---|---|---|---|
| pile length l [m] | 26 m | 26.4 m | 34.9 m |
| ultimate bearing capacity $R_1$ eq. top load | 20 MN | 52 MN | 53 MN |
| ultimate skin friction $q_{s,f}$ in the Cucaracha-Formation | | | |
| measured min. $q_{s,f}$ / max. $q_{s,f}$ | 80 kN/m²/ 560 kN/m² | 70 kN/m²/ 520 kN/m² | 65 kN/m²/ 450 kN/m² |
| $q_{s,f}$ used for design | 30 kN/m² / 100 kN/m² (1) | 100 kN/m² / 250 kN/m² (1) | 250 kN/m² |
| maximum tip resistance $q_{b,max}$ | | | |
| measured $q_{b,max}$ | - (2) | 7,500 kN/m² | 4,600 kN/m² |
| $q_{b,max}$ used for design | 2,000 kN/m (3) | 5,000 kN/m² | 400 kN/m² |

(1) limit skin friction varies along pile shaft.
(2) results for segment 3 could not be divided into skin friction and tip resistance.
(3) calculated tip resistance for displacement needed to mobilize limit skin friction.

*Moormann and Humpf* present an excellent and well-documented case history for the foundation design of the second bridge crossing the famous Panama Canal in Central America. The bridge is supported by shallow and pile foundations at the east and west side of the Canal, respectively. The paper focuses on three static pile load tests (T1, P2 and P3) involving single and multi-level testing using the Osterberg cell (or so-called O´cell) at the west side of the bridge. The subsoil conditions being mainly of volcanic debris are very complex and highly heterogeneous along the longitudinal axis of the bridge. Figure 17 shows the geology and arrangements of the single and multi-level tests on two 2-m diameter bored piles (T1 and P3). Prior to the load tests, there was no experience and knowledge about the performance of bored piles in this type of ground conditions. In total, three bored piles were tested. A summary of the test results are given in Table 3. By comparing with the test results from comparable diameter bored piles socked into decomposed granitic and volcanic rocks in Hong Kong (see Figure 18), it is interesting to note that the measured ultimate shaft resistance (or skin friction) and bearing capacity in the soft volcanic rock at the Canal are generally lower than those obtained in the weakest weathered rock (i.e., Grade III) in Hong Kong (Ng et al., 2001; Ng et al., 2004). It should be reminded that the decomposed Grade IV and Grade VI "rocks" are generally treated as "soils" worldwide. There is no doubt that the experience and test data gained from the bridge project at the Canal will benefit the geotechnical community significantly.

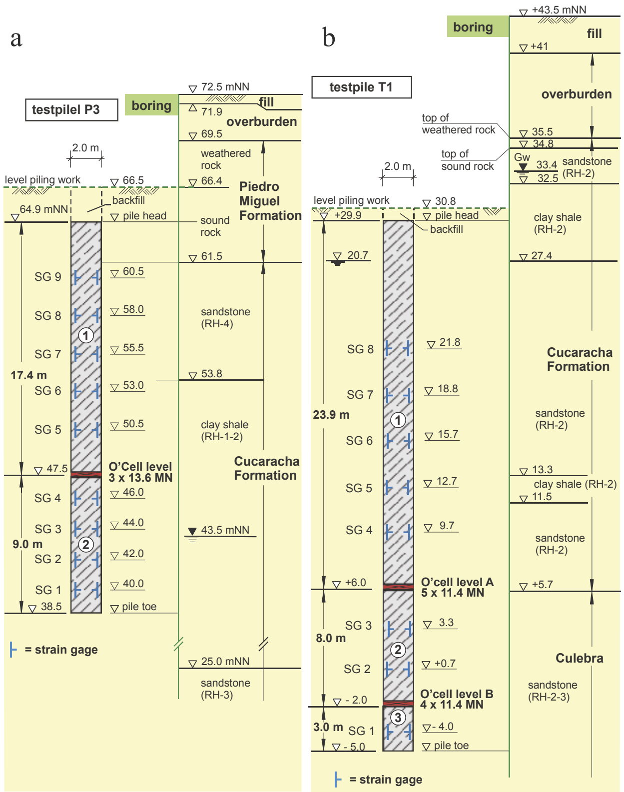


Figure 17. Single- and multi-level tests in the Cucaracha-Formation: test arrangement for test piles at axes P3 and T1 (Moormann and Humpf).

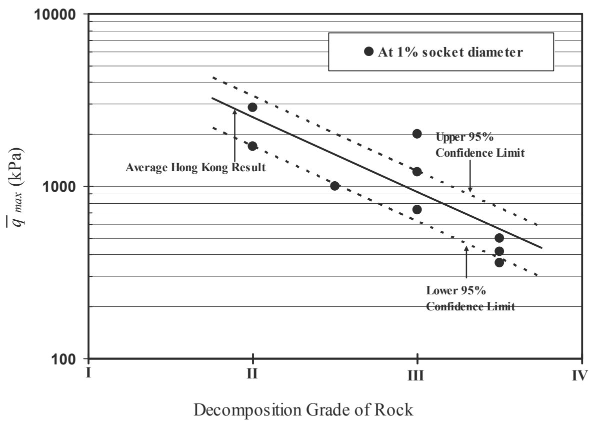


Figure 18. Maximum achieved side resistance vs. decomposition grade for Hong Kong granitic and volcanic rocks (Ng et al. 2001).

*Quick et al.* describe a foundation design case history for a 77m high building with a three-level underground carpark. The building is located in downtown Frankfurt where the ground conditions are highly heterogeneous. The design of the foundation is also complicated by the presence of two ground water tables including one unconfined and one subartesian ground water tables. With the assistance of 3D finite element analyses, a combined pile-raft-foundation scheme was designed for the project. Micro piles were installed at the bottom of the excavation to avoid hydraulic lift of the basement.

*Compagnucci* introduces and illustrates an underpinning system used to reduce building damages due to moisture changes in collapsing soils (i.e., porous fine sands and silty sands) in Argentine. It appears that small diameter floating bored piles (i.e., friction piles) are used. It is reported that a preloaded underpinning system has stopped the deterioration of building damages and assured good structural performance in three cities in Neuquén Province, Argentine.

*Lin and Woo* present computed results of a large number of 3D parametric numerical analyses to investigate the behaviour of piled raft foundation and possible effects of micro-pile installation on reducing lateral wall movements and ground settlements of a deep excavation at TIFC (Taipei International Financial Centre or the Taipei 101 project). The 101-story tall tower has a 22m deep basement. In the paper, they also report the results of 2D undrained creep simulations of the deep excavation.

In the 3D parametric analyses of the piled-raft interaction, the Taipei subsoil profile was simplified and simulated by an elastic-plastic model with Mohr-Coulomb failure criterion (or so called *M-C* model), whereas the piled-raft was simulated by a linear elastic (*L-E*) model. The required input parameters were back-calculated from static pile load tests carried out at the TIFC. They investigated the influence of various raft thickness *t* (1, 2, and 3*m*), pile spacing *s* (2*d*, 3*d* and 4.5*d*), where *d*=pile diameter=2*m*, pile group configurations (8×8, 5×5, and 3×3) and loading intensity *q* (1,000, 750, and 400 *kPa*) on the performance of piled-raft foundation in typical Taipei subsoil. It is not clear where the initial ground water table was in the analyses and it is not so obvious whether drained or undrained, effective or total stress analyses were carried out.

In the 3D numerical study of the effects of micro-pile installation on the performance of the deep basement, it is reported that groundwater flow calculation was incorporated in the 3-D stress and deformation analyses. Apparently effective stress parameters were used in the stress calculations. However, it is not entirely clear where the initial ground water table was and how the ground water flow calculation was coupled with the stress analyses.

*Reul and Ripper* describe an excellent case history of the foundation design for a 17-storey hotel, together with an adjacent 4-storey office building in Wuerzburg, Germany. The foundation level of the hotel is located about 4m below an artesian ground water table. The subsoil conditions are characterized by cavernous limestone. The caverns are the results of leaching in layers which contain gypsum or anhydrite. Any collapse of caverns and compression of residual soft soils may result in subsidence of ground surface and differential settlements.

Based on results of 3D finite element analyses of the subsoil-structure interaction and comparisons of technical advantages and economic of various design alternatives, a mat foundation was chosen together with a deep compaction grouting scheme to improve the Lower Leaching-zone at about 60m below ground in the vicinity of the hotel. The optimized design of the foundation comprised a monolithic raft with thickness varying between 2m in the area of the hotel and 0.8 m in the area of the office building. To provide sufficient uplift resistance of the foundation, approximately 100 ground anchors were installed in areas with small surcharge loads from the superstructure. Very interesting field measurements of total dead weights, uplift force due to ground water pressure and settlements of the raft foundation during construction are reported in the paper.

*Boyko et al.* appear to report a procedure of analyzing the interaction of high-rise building and its foundation numerically. Both static and dynamic loading conditions seem to be considered and included in the numerical analyses.

*Isobe and Kimura* investigate performance of bridge caisson foundation reinforced by steel pipe sheetpiles (SPSP). Centrifuge model tests and 3D elasto-plastic FE analyses were carried out. Figures 19 and 20 show a cross-section and plan view of the model foundation, respectively. A total of 20 centrifuge model tests were conducted without and with SPSP and three fixing types were considered between the caisson and the surrounding SPSP. Good quality of test data is presented. However, it is not so obvious how the loading was applied in the tests. In the back-analysis of the centrifuge model tests, the dry dense Toyoura sand was modelled by the so-called $t_{ij}$ sand model. Consistency between the measured and computed load-displacement relationships is illustrated. It is not obvious, however, whether soil-wall interface was simulated or not. In the paper, a series of 3D FE parametric analyses using the Drucker-Prager soil model are also carried out to investigate and verify the applicability of the SPSP for an existing caisson founded in water. It is not clear why the $t_{ij}$ sand model used for calibrating the centrifuge tests is not adopted to study this existing caisson problem. In the paper, the authors conclude that connection details between caisson and SPSP are very important and flexural rigidity should be considered carefully in design.

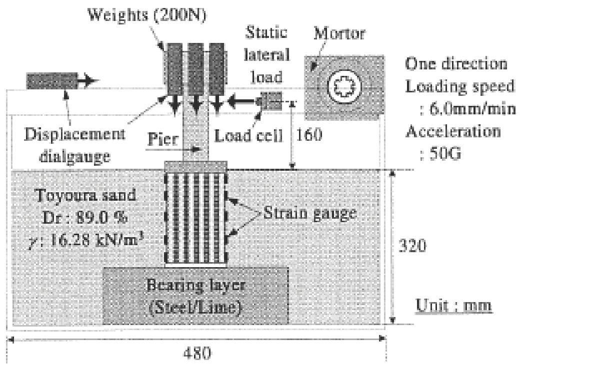


Figure 19. A cross-section of the model foundation (Isobe and Kimura).

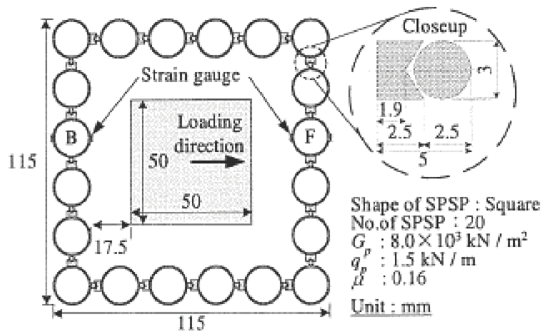


Figure 20. Plan view of the model foundation (Isobe and Kimura).

## 6 CONCLUDING REMARKS

The papers in this session provide some very interesting and excellent design case histories and results of field monitoring on various soil-structure interaction problems. There is no doubt that these case histories and field data will advance and improve our knowledge in the areas of soil-structure interaction. In this session, some authors, who have presented the comparisons of Class-A predictions and corresponding field measurements, should be applauded.

It is evident that more and more advanced in-flight construction simulation tools and novel modelling techniques such as the 4-axis robotic manipulator, the inclined loading device and the expanding tool are used to investigate various geotechnical problems in centrifuge model tests. High quality centrifuge data are presented in many papers in this session. Not only the test data can assist us to improve our understanding of the mechanisms involved but also they are essential for calibrating constitutive soil models and numerical modeling procedures.

On the other hand, it is quite disappointing to observe that fundamental differences between drained and undrained analyses are not properly discussed and modelled in some papers. Moreover, distinctions between effective stress and total stress analyses together with appropriate model parameters are not correctly differentiated and adopted. In a number of cases, ground water table is not defined in some FE analyses. Without proper explanations and justifications of input parameters, good "matches" between measured values and back-analyzed results (i.e., Class-C predictions) are reported. These good "matches", however, have very limited scientific values.

## ACKNOWLEDGEMENTS

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## LIST OF PAPERS REVIEWED IN SESSION 2C

Ali Jawaid, S.M., Madhav, M.R. Continuum approach for analysis of short composite caisson foundation.
Ameratunga, J., Shaw, P., Beohm, W.J., Boyle, P.J. Seawall construction in Moreton Bay, Brisbane.
Blackburn, J.T., Sylvester, K., Finno, R.J. Observed bracing responses at the Ford Design Center excavation.
Boyko, I., Saharov, O., Nemchynov, Yu. The peculiarities of stress-strain state at interaction of high rise buildings and structures with the base.
Bustamante, M., Bourgeois, E., Gianeselli, L., De Justo, J.-L. The foundations of the 2nd railway bridge of Argenteuil.
Chua, H.Y., Bolton, M. Modeling of horizontal earth pressures on retaining walls.
Compagnucci, J.P. Underpinning of foundations in collapsing soils.
Eslami, A., Gholami, M. Bearing capacity analysis of shallow foundations from CPT data.
Gebreselassie, B., Kempfert, H.-G. Mobilization of the earth resistance of a normally consolidated cohesive soils.
Gong, J.F., Huang, X.L., Teng, Y.J. Rigidity characteristic and deformation calculation of large-area thick raft foundation.
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Ilyichev, V.A., Konovalov, P.A., Nikiforova, N.S. Deformations of the buildings located near foundation trenches and underground excavations and the measures for their reduction.
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Marten, S., Delattre, L., Pioline, M., Vinceslas, G., Joignant, Ph., Lavisse, J. Observed behaviour of a quay wall at the new 'Port 2000' at Le Havre, France.
Mecsi, J., Gokalp, A., Duzceer, R. Densification of hydraulic fills by vibroflotation technique.
Moormann, Ch., Humpf, K. Foundation design for a new cable-stayed bridge crossing the Panama Canal.
Ng, C.W.W., Lee, C.J, Xu, G.M., Zhou, X.W. Novel centrifuge simulations of restoration of building tilt.
O'Brien, A.S., Hardy, S., Farooq, I., Ellis, E.A. Foundation engineering for the UK's new national stadium at Wembley.
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