23rd GETD AGM Talk on "Centrifuge and Numerical Investigation of Pile Performance Subjected to Stress Relief Due to Deep Excavation" by Prof. Charles W. W. Ng (HKUST) Reported by Ir. Liew Shaw Shong

It was the honour of Geotechnical Engineering Technical Division of IEM having Prof. Charles W. W. Ng from The Hong Kong University of Science & Technology (HKUST) to deliver a technical talk at Tan Sri Prof. Chin Fung Kee Auditorium, Wisma IEM on 9 June 2012. The talk was chaired by Ir. Liew Shaw Shong, the present technical division chairman and was attended by 115 participants.

The presentation started with a brief introduction of the geotechnical testing facilities in HKUST and followed with the fundamental principles of geotechnical centrifuge modelling with schematic illustration on how rotating a test sample generating a linearly increasing centrifuge stress within the test sample to study the soil behaviours (dilative below critical state line and contractive above critical state line) as shown in Figure 1.

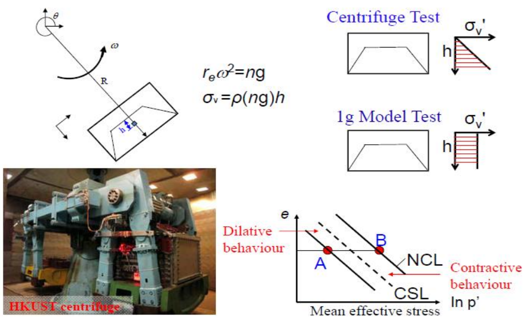


Figure 1: Fundamental principles of centrifuge modelling

The following four principal applications of centrifuge technology are summarised below:

a. **Modelling of Prototypes** such as slopes, piles, tunnels, excavations, geo-environmental & earthquake-induced problems, consolidation settlements.
b. **Investigation of New Phenomena** such as explosions, plate tectonics, liquefaction problems, contaminant transports
c. **Parametric Studies** such as bearing capacity of footings on slopes, laterally loaded pile groups
d. **Calibrations of Numerical Models and Methods** - When there is a good match of the computer numerical modelling to the field observation, what does this really imply? The following questions arise:
    - Is the numerical model, simulation procedure or the model parameters used correct?
    - A converged solution is found?
    - Is our understanding of the mechanical behaviours and mechanism improved?

With the uncertainties in the ground conditions (such as non-homogeneity of the sub-soils, existence of cross-anisotropy, perched groundwater table, coefficient of lateral earth pressure at rest, zero displacement boundary, permeability and degree of saturation in sub-soils, etc), which might not be fully captured in the numerical model, the good matching result would be highly possible just a sole coincident bearing no true value to the problem. Hence, the direct calibration of numerical modelling to actual field observation can be a dangerous practice. However, physical model test using 1g prototype model and centrifuge model test with known boundary conditions and ground conditions can be constructed to simulate the actual site conditions and verify both the numerical modelling and also the actual field observation from case history, hence filling up the missing link between numerical modelling and field observation.

In most pile foundation design, it is very common to perform a static maintained load test at the ground level. If the project has deep basement structure, a pile sleeve is usually provided along the pile shaft above the lowest basement level to prevent stress interaction from the embedded soils. However, the actual foundation piles with a deep basement often work underneath the basement structure to provide support to the columns. With the basement excavation, the actual stress in the subsoils embedding the piles below the basement would have been relaxed as a result of stress relief. As such the effect of stress relief on pile capacity and stiffness has seldom been examined when interpreting the static pile load test conducted at ground level well above the basement. Figure 2 illustrates the questionable representation of the conventional single test pile programme at ground level and at basement level to the actual pile working conditions.

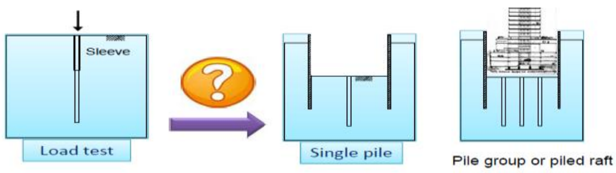


Figure 2: Representation of conventional load tests at ground level with pile sleeve and beneath deep basement, and pile group or pile raft in service below deep basement

Relevant clauses in Eurocode 7 (BS EN 1997-1:2004) on pile verification tests are extracted :

- Clause 7.4.1 (1) - The design shall be based on one of the following approaches :
    - the results of ***static load test***;

- empirical, analytical ***calculation methods, dynamic load test*** whose ***validity*** has been demonstrated by ***static load tests***;
- the observed performance of a comparable pile foundation

- Clause 7.5.1 (4) - If pile load test is carried out, it shall normally be located where the ***most adverse ground conditions*** are believed to occur.

The research objectives in this series of centrifuge model tests aim to (a) study the capacity of single piles with and without stress relief due to basement excavation; (b) understand and quantify the governing mechanisms for shaft resistance of pile in both non-dilatant and dilatant soils; and (c) to investigate the capacity, efficiency and failure mechanism of 3x3 pile group with stress relief.

In order to characterise the soil-structure interface, normalised roughness ($R_n = R_{max}/D_{50}$) as shown in Figure 3 is used to determine the two possible failure mechanisms at the soil-structure interface (Fioravante, 2002) as below.

- If $R_n < 0.02$: non-dilatant interface – particle sliding along the interface
- If $R_n > 0.10$: dilatants interface – failure happens within a shear band in the soil

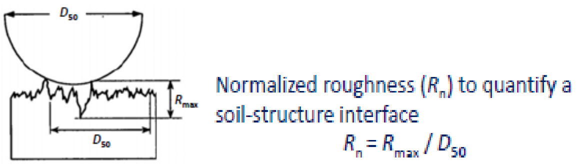


Figure 3: Normalised roughness, R_n (Kishida & Uesugi, 1987)

The photographic visualisation of two response types at soil-structure interface can be illustrated in Figure 4.

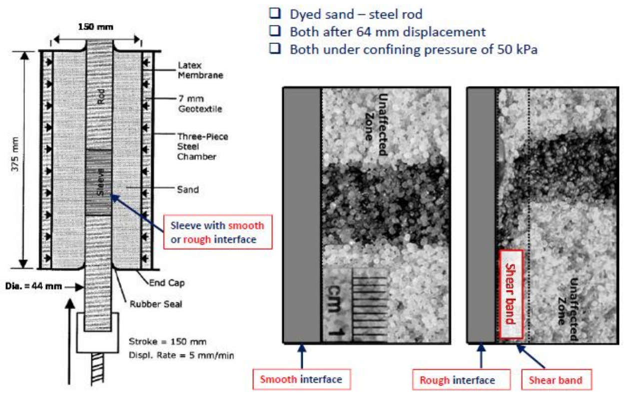


Figure 4: Response of soil-structure interface

To research the effect of the stress relief to pile capacity, four scenarios of the pile load testing condition as summarised below were explored in order to compare their performance:

A. Single pile tested at ground prior to excavation resembling most of the conventional load tests, which are conveniently implemented at site before production pile installation. Sometimes, pile sleeve is introduced to eliminate interference of soil friction above the actual pile cut-off level.
B. Single pile subjected to stress relief due to excavation.
C. Elevated pile group (3x3) with stress relief and no contact between the pile cap to the soil platform for load transfer other than the piles.
D. Pile group (3x3) with stress relief, but the pile cap is in contact with soil platform enabling load transfer from pile cap to the soil platform.

Figure 5 visualises the abovementioned testing programme with smooth pile shaft simulating low friction pile in non-dilatant soils and rough pile shaft interface simulating high friction pile in dilatant soils. To increase the pile-soil interface roughness, the pile shaft surface was coated with epoxy and sand grains.

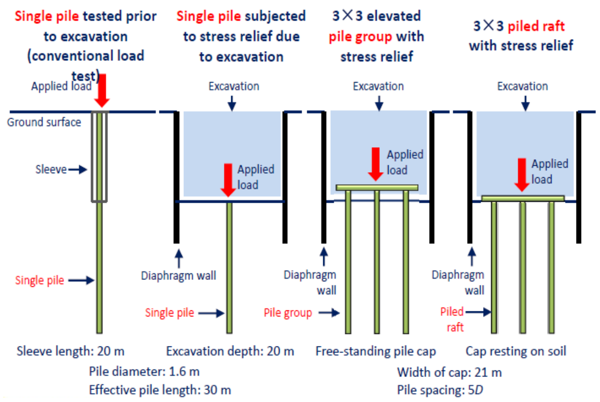
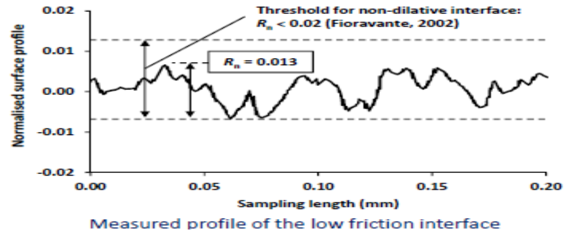


Figure 5 : Test Programme of Centrifuge Model Testing

Roughness of pile-soil interface encourages rolling of soil particles over each other rearranging themselves to a less compact state, hence presenting a dilative behaviour of soil in shearing. The soil dilation within the shear band will induce additional effective stress, thus increasing the frictional resistance when subject to shearing as presented in Figure 6.

To verify the soil dilatancy at pile-soil interface, modified direct shear box tests using bottom aluminium plates with both smooth surface and epoxy coated with sand grains were carried out. Toyoura sand was compacted with relative density of 65% as the model soil. The shearing test results in Figure 7 show comparison of stress ratio and the dilative displacement in the high friction surface and the low friction surface.

In preparing the 3x3 pile group model, a 20mm thick aluminium plate adapted to nine aluminium model piles of 16mm diameter spaced with pile centre-to-centre spacing of five times of pile diameter. As the cap-soil stiffness ratio, k_{rs}, is larger than 25, the pile cap can be considered as rigid pile cap. The centre pile, corner pile and edge pile were equipped with instruments for load transfer measurement of pile axial load along the pile shaft with different imposed load in flight. Figure 8 shows the 3x3 pile group test configuration.



	Low friction piles	High friction piles
Roughness R_n	0.013 (<0.02*)	0.21 (>0.10*)
Objective	Piles in non-dilatant material (normally consolidated clay / loose sand)	Piles in dilatant material (overconsolidated clay / dense sand)
Shaft resistance	$\tau_p = \sigma'_n \tan\delta$	$\tau_p = (\sigma'_n + \Delta\sigma'_n)\tan\delta$ (Boulon and Foray, 1986; Houlsby, 1991)

*Note: thresholds for R_n from Fioravante (2002)

Figure 6: Measured profile and mechanism of pile-soil interface

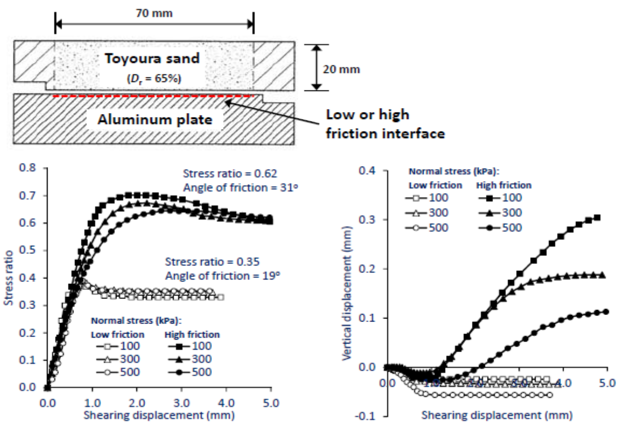


Figure 7: Direct shear box test simulating pile-soil interface

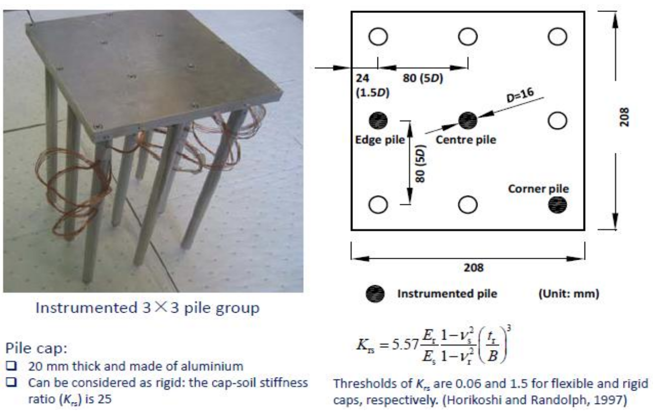


Figure 8: Typical test arrangement of instrumented pile group

In this series of centrifuge model testing, the effect of excavation is simulated by draining out of the heavy fluid contained in the space modelling the basement. The setting up and testing procedures of the centrifuge tests for both (a) Single pile tested prior to excavation as in the conventional load test, and (b) Single pile subjected to stress relief due to excavation are illustrated in Figure 9. The amount of stress relief to the soil platform beneath the basement will be equal to the centrifuge weight of the heavy fluid drained out. Loading of 100N (10kN in prototype based on scaling laws for force) was applied incrementally to the model pile during the flight.

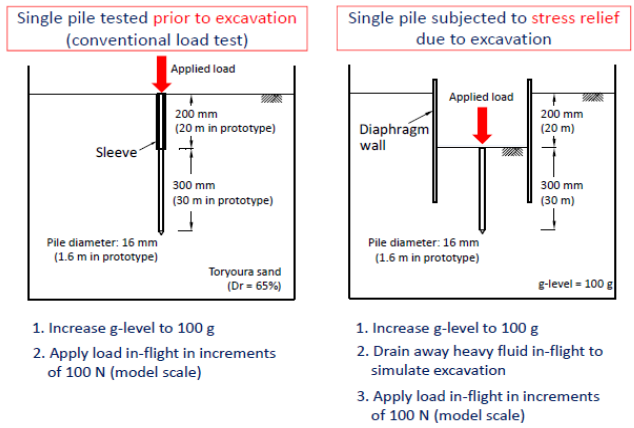


Figure 9: Testing procedures for single pile tested prior to excavation and subjected to stress relief due to excavation

Figure 10 shows the comparison of load-settlement curves for the single pile in a non-dilatant soils with pile sleeve (without stress relief) and with stress relief due to 20m deep excavation. Considering two failure criteria suggested by Ng et al (pile load at settlement of 0.045D+0.5(PL)/(AE)) and Eurocode 7 (pile load at settlement of 10% of pile diameter), the measured pile capacity with stress relief is 20% and 16% lower than that without stress relief corresponding to the two failure criteria respectively.

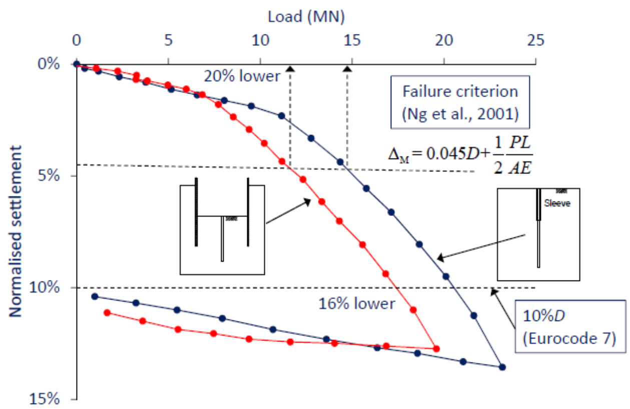


Figure 10: Load-settlement curve of in-flight load tests on single piles in non-dilatant soils

As for the measured pile axial load profiles for single pile with and without stress relief, it was observed that the pile with stress relief generally has lower overall pile capacity as shown in Figure 11. Similarly, the lower rate of reduction of axial load along the pile with stress relief is less than the pile without stress relief implying lower shaft resistance in the pile with stress relief, but more pile base resistance is mobilised in the pile with stress relief.

The excavation geometry (R/H) and ratio of excavation depth to pile length (H/L) have significant influence in pile capacity, the higher in either of the parameters, more reduction in the pile capacity. For the tests conducted, the measured reduction in pile capacity is in good agreement with the FEM parametric study by Zheng, Diao and Ng (2011) as shown in Figure 12.

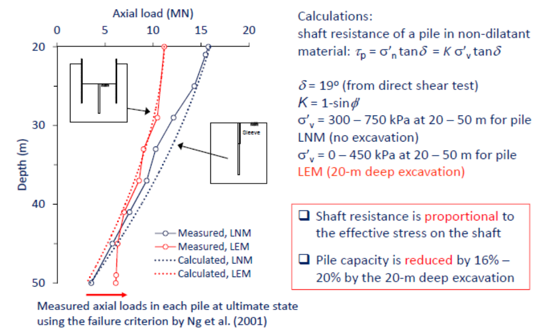


Figure 11: Pile axial load profile of in-flight load tests on single piles in non-dilatant soils

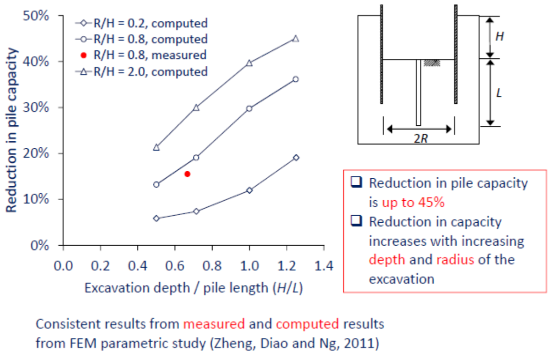


Figure 12: Effect of excavation geometry and pile length

However, the performance of single pile in dilatant soils with stress relief in Figure 13 shows stiffer load-settlement curve leading to higher interpreted pile capacity based on the two failure criteria. From the pile axial load profiles in Figure 14, it is not surprised to observe that the enhancing effect of pile shaft resistance in dilatant soils even with the counteracting pile capacity reduction effect from the stress effect due to excavation. The increased effective normal stress around the pile shaft due to soil dilatancy during shearing has increased pile shaft resistance as evidenced by a rapid reduction in pile axial load profile (the right figure) in Figure 14.

Figure 15 shows the interpreted unit shaft resistances mobilised at every instrumented segment of pile shaft with the two stress conditions (without and with stress relief). The soil dilatancy effect in the changes of effective normal stress seems to be more dominant than the stress relief effect from excavation.

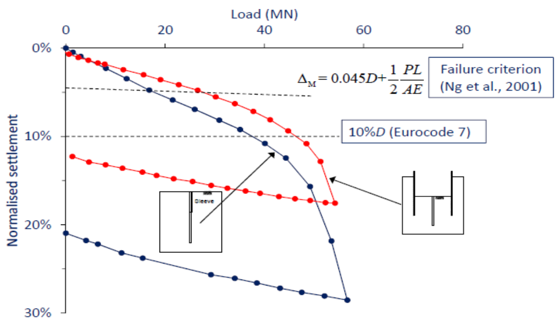


Figure 13: Load-settlement curve of in-flight load tests on single piles in dilatant soils

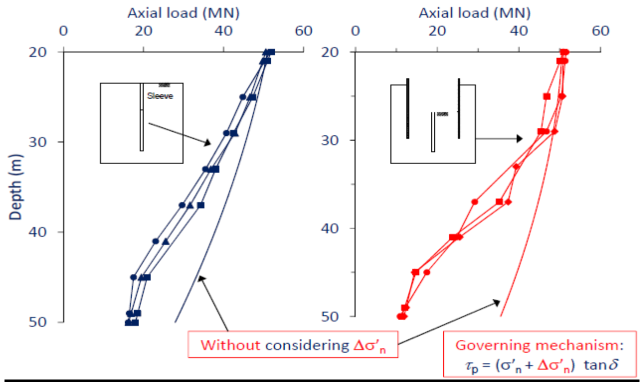


Figure 14: Pile axial load profile of in-flight load tests on single piles in dilatant soils

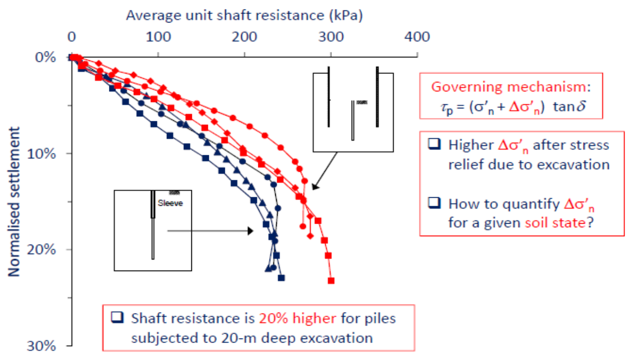


Figure 15: Mobilisation of pile shaft resistance

When the soil dilates during shearing, the normal stress on the pile-soil interface does not remain constant. Three boundary conditions for the pile-soil interface and the resulting shear resistances are shown in Figure 16. To study this dilative behaviour, Discrete Element Method (DEM) numerical model with constant normal stiffness (CNS) boundary condition was used to simulate the cavity expansion behaviour relating to the outward displacement (Δy) of the pile soil interface and the increase of normal stress ($\Delta\sigma'_n$) in a parametric study.

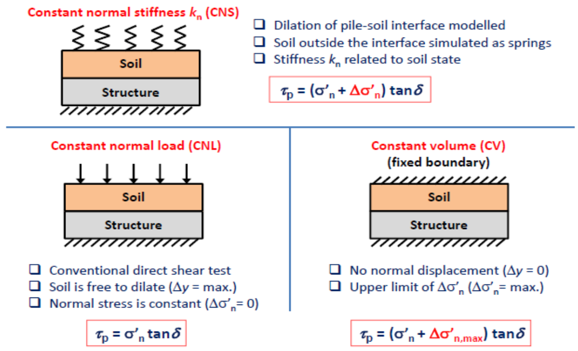


Figure 16: Boundary conditions for dilative pile-soil interface

The numerical results with CNS boundary condition in Figure 17 revealed that the mobilised stress ratio curve with stress relief (initial normal stress of 400kPa unload to 100kPa prior to shearing) showing the peak value of about 1.0 (at 2% shear strain) before softening to a stress ratio of 0.72 (at about 10% shear strain) as compared to the result without unloading. It is evidenced that the soil dilatancy effect overwhelms the stress relief effect resulting in consistently higher normal stress increase during entire shearing process.

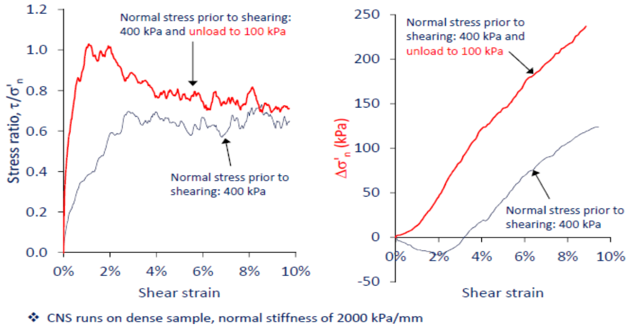


Figure 17: Effect of stress relief on pile-soil interface response

The comparisons of the computed theoretical unit shaft resistance and pile axial load profile considering the changes in effective normal stress to the measurements in the centrifuge test results presented in Figures 18 and 19 show reasonably good agreement.

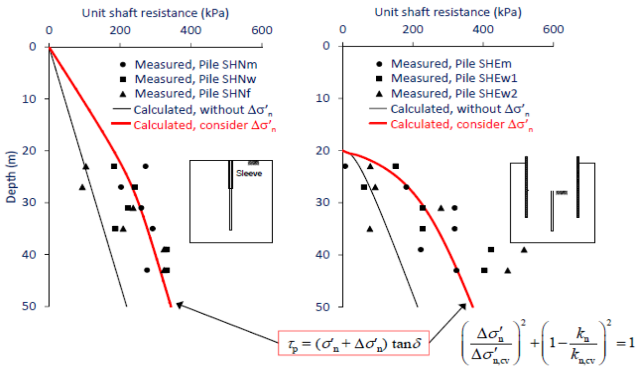


Figure 18: Calculated and measured shaft resistances

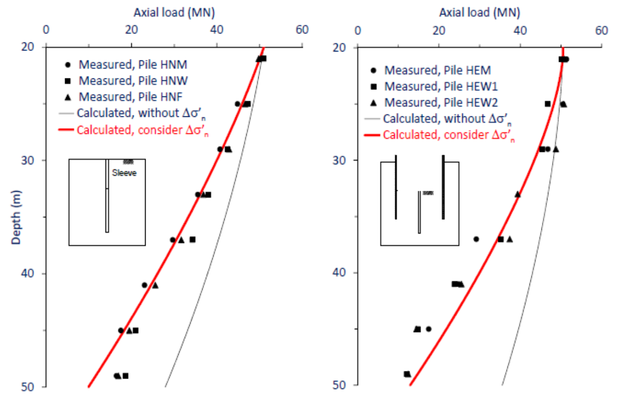


Figure 19: Calculated and measured pile axial load distributions

From the above findings, it can be observed that a conventional pile load test in non-dilatant material is not conservative. Hence the most adverse condition for load test as required in Eurocode 7 (BS EN 1997-1:2004 Clause 7.5.1) shall be at the end of excavation. If this is not possible, a reduction in pile shaft resistance proportional to stress relief should be considered in the calculation. However, for pile in a dilatant material, the most adverse condition may occur either prior to excavation or after it, due to two counteracting effects on changes in normal effective stress. Though there is reduction in effective normal stress due to stress relief, the effective normal stress can increase significantly as a result of dilation during shearing. In this case, it is safe to ignore beneficial increase of pile capacity due to soil dilatancy in design.

Finally, the test results for pile capacity and observed failure mechanisms of elevated pile group subject to stress relief are presented in Figures 20, 21 and 22. Based on the failure criterion of pile settlement of 10% pile diameter as in Eurocode 7, Figure 20 shows the interpreted group pile capacity of 293MN, which is equivalent to 33MN per pile. Comparing with single pile capacity of 45MN with stress relief condition, it was deduced that the group efficiency is approximately 0.7. From the load-settlement curves of single pile and group piles, the single pile appears to have stiffer load settlement performance. When examining the unit shaft resistance of the single pile and group piles in the separating soil embedment zones, namely upper half and lower half as shown in Figure 21, the unit shaft resistances of both single pile and group piles are consistently higher in the lower half and that of the single pile is always higher than that of the group piles for the corresponding soil embedment zones. Mobilised unit shaft resistance along the upper half of each pile is only about 40% of the single pile at settlement up to 15% of pile diameter in this study. The normalised shaft resistance in upper half tends to be fairly constant (about 0.4) with increasing normalised pile settlement whereas, for the lower half, the normalised shaft resistance shows increasing trend with increasing normalised pile settlement.

Figure 22 shows the pile axial load profile of a sleeved single pile (without excavation) at the interpreted pile load with the failure criterion in Eurocode 7 corresponding to a conventional load test result. With the same Eurocode 7 failure criterion to the pile group centrifuge test simulating the pile group test (which is usually not performed in normal piling engineering practice), the interpreted pile group capacity of 293MN is presented in Figure 20. In the normal pile group assessment from the result of single pile load test, there are two following practices suggested in Eurocode 7 (Clause 7.6.2.1(3)): ***(a) Failure of the pile individually -*** Assuming individual pile in the pile group has same unit shaft and toes resistance as in the single pile, the computed pile group capacity would be 342MN (9 piles x 38MN/pile), which overestimates pile group capacity by 17%. ***(b) Failure of piles and the soil between them acting as a block -*** Assuming the pile group block has the same shaft and toes resistance as in those of the single pile, the computed pile group capacity would be 1497MN, which overestimates the pile group capacity by about 400%. As the test shows lower pile group capacity than the computed group capacity, there are likely other mechanisms other than the two aforementioned extreme cases. Hence, the computed pile group capacity is not conservative and has to be used with care.

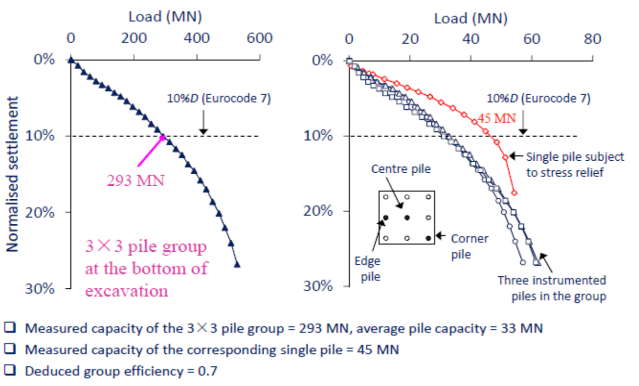


Figure 20: In-flight load test on elevated pile group subjected to 20m deep excavation in dilatants soil

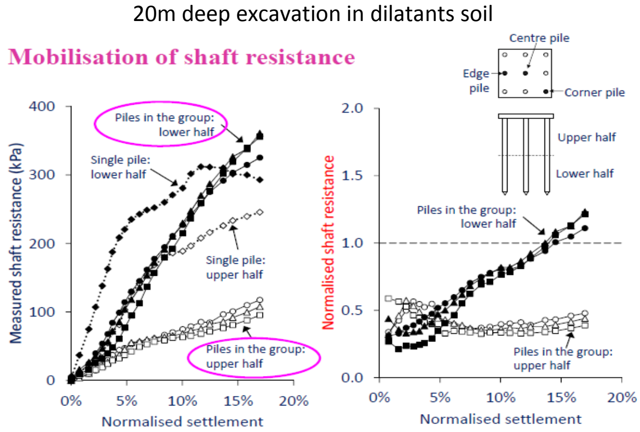


Figure 21: Mobilisation of pile shaft resistance of pile group

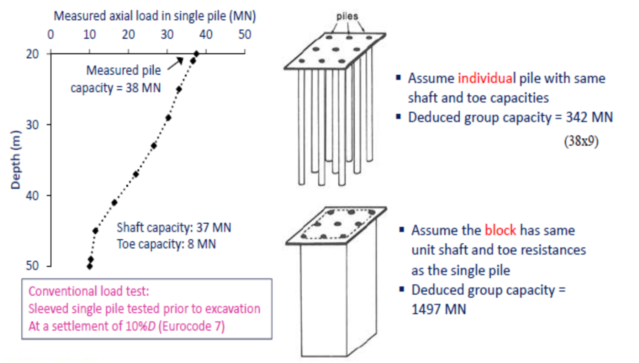


Figure 22: Derivation of pile group capacity and critical failure mechanism

Summary of Key Findings

(A) Single pile in non-dilatant material:

- Limit shaft resistance : $\tau_p = \sigma'_n \tan\delta$
- Pile capacity is reduced by 20% when subjected to stress relief of 20m deep excavation. Sometimes reduction in pile capacity may be up to 45%, depending on excavation geometry and pile length.
- Conventional load test in non-dilatant soils is non-conservative (even with pile sleeve)
  - The most adverse condition for a load test is to conduct the load test at the bottom of basement after the excavation.
  - If this is not possible, reduction in shaft resistance proportional to stress relief should be considered for actual pile performance in service.

(B) Single pile in dilatants materials:

- Limit shaft resistance : $\tau_p = (\sigma'_n + \Delta\sigma'_n)\tan\delta$
- Reduction in σ'_n is proportional to stress relief, but the magnitude of $\Delta\sigma'_n$ increases by 30% when subjected to stress relief of 300 kPa for the piles
- The most adverse condition may occur either prior to excavation or after it, due to the two counteracting terms, namely the stress relief factor and the soil dilatancy during shearing:
  - Reduction in σ'_n due to stress relief should be considered
  - But also increase in $\Delta\sigma'_n$ due to dilation and yields conservative result; may be ignored in design

(C) 3x3 pile group capacity and failure mechanism:

- Based on the settlement criterion of 10%D (Eurocode 7): Measured capacity of the pile group = 293 MN (single pile = 45 MN) and Group efficiency = 0.7
- Mobilised pile shaft resistance along upper half of each pile is only 40% of the single pile at settlement up to 15% of pile diameter in this study
- The two failure mechanisms by Eurocode 7:
  - Assuming individual pile failure, overestimates capacity by 17%
  - Assuming block failure, overestimates capacity by 400%
- Mechanism other than the two extreme cases exists, capacity may be lower than either case and is non-conservative

During the Q&A session, there were active discussions from the floor over the subject matter and mutual exchange of opinions. The technical talk ended at 7:00pm with the presentation of mementoes by GETD's past chairman, Ir. Mun Kwai Peng to Prof. Charles Ng as shown in Figure 23.

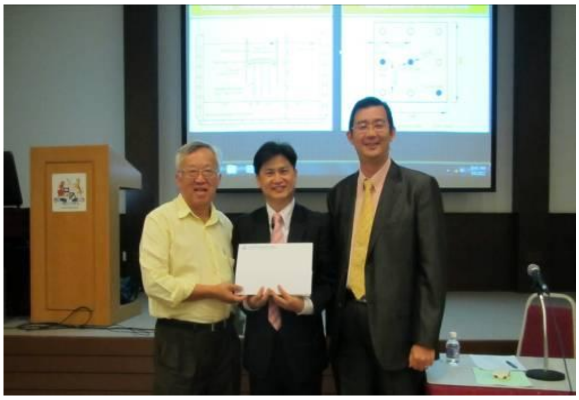


Figure 23: Presentation of mementoes to Prof. Charles Ng
(l-r: Ir. Mun Kwai Peng, Prof. Charles Ng and Ir. Liew Shaw Shong)

Acknowledgment: The author wishes to express great gratitude to Prof. Charles Ng in providing the presentation slides for preparation of this report and also the permission granted to upload the presentation slides to IEM website for members' reference.