Think Beyond Standard Code of Practice Rules
Mobilizable Strength Design

By Ir. Chua Chai Guan

It is with deep sadness that we advise readers of the sudden passing of the author in July 2013. The late Ir. Chua Chai Guan was the former Secretary and Treasurer of Geotechnical Engineering Technical Division in the IEM. He had worked in the geotechnical field for 15 years in Malaysia as well as in Hong Kong, Singapore, Bangkok and Ho Chi Minh City. He was the principal of Substrata Engineering Sdn Bhd, a contracting company offering geotechnical solutions to clients.

The one day workshop entitled "Performance-based Design in Geotechnical Engineering" was conducted by Prof Malcolm Bolton and Prof Charles W.W. Ng on 9 December (Sunday) 2012 at the Auditorium Tan Sri Prof. Chin Fung Kee of Wisma IEM. It was attended by about 61 participants. This workshop was supported by the Institution of Civil Engineers (ICE), Malaysia, Southeast Asian Geotechnical Society (SEAGS) and Association of Geotechnical Societies in Southeast Asia (AGSSEA).

Prof. Malcolm Bolton is the Professor of Soil Mechanics at Cambridge University, and has been Director of the Schofield Centre for Geotechnical and Construction Modelling since 1995. He is a Fellow of the Royal Academy of Engineering and has won awards from the Institution of Civil Engineers, the Institution of Structural Engineers, the British Geotechnical Association and the Canadian Geotechnical Society. With the assistance from Prof Charles Ng of Hong Kong University of Science and Technology, the GETD managed to secure the presence of Professor Bolton for this workshop amidst his busy schedule during the course of his tour of Asian countries delivering the 52nd Rankine Lecture.

Prof. Malcolm Bolton firstly provided an overview of small strain stiffness of soil. He assessed the relationship between normalised shear modulus (G/G_o) and shear strain from the 520 selected static and dynamic tests of coarse-grained and fine-grained soils. G is the secant shear modulus at any strain. G_o is the elastic shear modulus at very small strain (G at γ =0.0001%). The observed relationship could be predicted using the modified hyperbolic equation $\frac{G}{G_o} = \frac{1}{\left[1+\left(\frac{\gamma}{\gamma_r}\right)^{a}\right]}$ where γ_r is reference strain value at which G/G_o = 0.5 and a is called the curvature parameter. Both of them could be correlated to simple soil properties as follows : a = 0.74 and $\gamma_r = 1.25 W_L 10^{-4}$ for clays and $a = Uc^{-0.075}$ and $\gamma_r = 8\, e\, I_D 10^{-4} + U_C^{-0.3} p' 10^{-6}$ where W_L =liquid limit, U_c =uniformity coefficient, e = void ratio, I_D=relative density and p'= effective mean stress. The reliability of prediction has a factor of ±1.3 (2 std. dev.).

Interestingly, the rate of deterioration of stiffness with strain was similar for clays and sands as pointed out by Prof. Bolton.

He shared that the quasi-hyperbolic backbone curve, its hysteresis loops due to cycling, the "S-shaped curve" of G/G_0 versus $\log \gamma$, were all reflections of the development of a strong force network revealed by Discrete Element Mechanics (DEM).

He went further to present the observed relationship between the degree of strength mobilization (1/M) and shear strain from 115 triaxial tests on clays [see Figure 1].

The trend line could be approximated using a power curve as follows:-

$$\frac{\tau_{mob}}{c_u} \approx 0.5 \left(\frac{\gamma}{\gamma_{M=2}}\right)^b$$

where $\gamma_{M=2}$ is mobilised strain at $0.5c_u$ and *b* is a coefficient related to over consolidated ratio (OCR). The tests on Kaolin clays revealed that *b* = 0.011(OCR) + 0.371 and $\log_{10}(\gamma_{M=2}) = 0.680\log_{10}(\text{OCR}) - 2.395$. He also verified the above approximation for London clays from triaxial and self-boring pressure meter (SBP) tests. Typically, the stress history of a stratum would offer increase of c_u and reduction of $\gamma_{M=2}$ with increasing depth.

Before the lunch break, Prof. Bolton stirred up the audience by pointing out that what had been taught in the conventional textbooks and standard Code of Practice (EC7) for shallow foundation in clays was wrong in terms of control of settlement by applying a factor of safety of 2 to 3, deformation based on oedometer stiffness, specification based on limit stage design framework of ultimate limit state (ULS) and serviceability limit state (SLS).

Prof Bolton presented Asraf Osman's deformation mechanism for a shallow foundation, then utilised the Mobilised Strength Design (MSD) to capture its load-settlement behaviour. The accuracy of MSD method in estimating settlement was as good as that predicted by the finite element method using an advanced constitutive soil model. Thus the curve of mobilised strength versus shear strain exhibited in undrained triaxial test could be used for predicting settlement. This was validated through field tests as well.

He then showed the footing settlement pattern observed in centrifuge tests by Brendan McMahon [see Figure 3]. The 4 phases of settlement are undrained indentation, undrained creep, continuing creep with additional consolidation and drained creep at a reduced rate after consolidation is nearly completed. He excited the floor by stating that what had been observed in an oedometer was completely misleading. The creep settlement would take place from the undrained stage. Based on the theory of elasticity, the consolidation settlement could be 1.4 times of undrained settlement.

He examined the limit state of footing using MSD in the light of allowable differential settlement ($\text{SLS}:\Delta w=1/1000$ for onset of cracks and ULS:Δw 1/400 for gaping cracks). In order to achieve the desirable differential settlement, the required mobilised factor, M (c_u/τ_{mob} or safety factor) is 3.8 and 1.46 for SLS and ULS states, respectively. This was much greater than the conventional FOS of 2 to 3 for shallow foundations. He stressed that the soil deformed a lot prior to failure.

Prof Charles Ng took over the floor in the afternoon. He presented a series of centrifuge test programmes which modelled twin tunnelling effects on pile and pile group by his research students. The scenario considered were the 4 probable tunnelling sequences (TT, SS, ST and TS) of twin tunnels bored through near the locations of pile toe (T) and shaft (S). The clear distance between the single pile and tunnels was 1.52m. The apparent loss of pile capacity (ALPC) was benchmarked with the reduction of capacity to the pile capacity at FOS=1.5 based on the failure criterion proposed by Ng *et al* (2001). The tests showed that APLC could be as high as 36%. The induced pile head settlement by ST was found about 30% higher than that by TS. The induced total ground movement was about the same by TS and ST.

The pile group modelled in the centrifuge tests were four numbers. The tunnelling sequence for twin tunnels was G-ST, G-SB, G-TT and G-BB, where G, S, T and B respectively denote pile group, pile shaft, pile toe and pile base. The clear distance between tunnels and the pile group was 2.2m and the clear distance between the pile toe and the crown of tunnels was 3m. The results showed that ALPC in pile group of G-ST was smaller than that of G-SB but the transverse tilting of pile cap was 67% higher. He highlighted that the tilting of pile cap in the longitudinal direction of tunnels was 10 times higher than that of the transverse direction for both G-ST and G-SB. For G-TT, the transverse tilt is more critical during construction than that of end of twin tunnelling but this was vice versa for G-BB.

The last part of the workshop was a forum session by the speakers and 4 invited panellist namely Ir. Tan Yang Kheng , Ir. Yee Thien Seng, Ir. Liew Shaw Shong and Ir. Dr. Chin Yaw Ming and the moderator, Ir. Dr. Chan Swee Huat. It meant to engage the audience to exchange opinions with the speakers and panellists on the existing ways of handling ground investigations, choice of design parameters, numerical procedures, adoption of a Code of Practice, predictions of deformation and risk management. There was a long and heated discussion on the awareness of an "apparent" slope failure mechanism and the level of risk-taking approach in managing hill-site development. The workshop ended at with the presentation of tokens of appreciation from the organiser to the speakers and which was accompanied by huge applause from the floor.

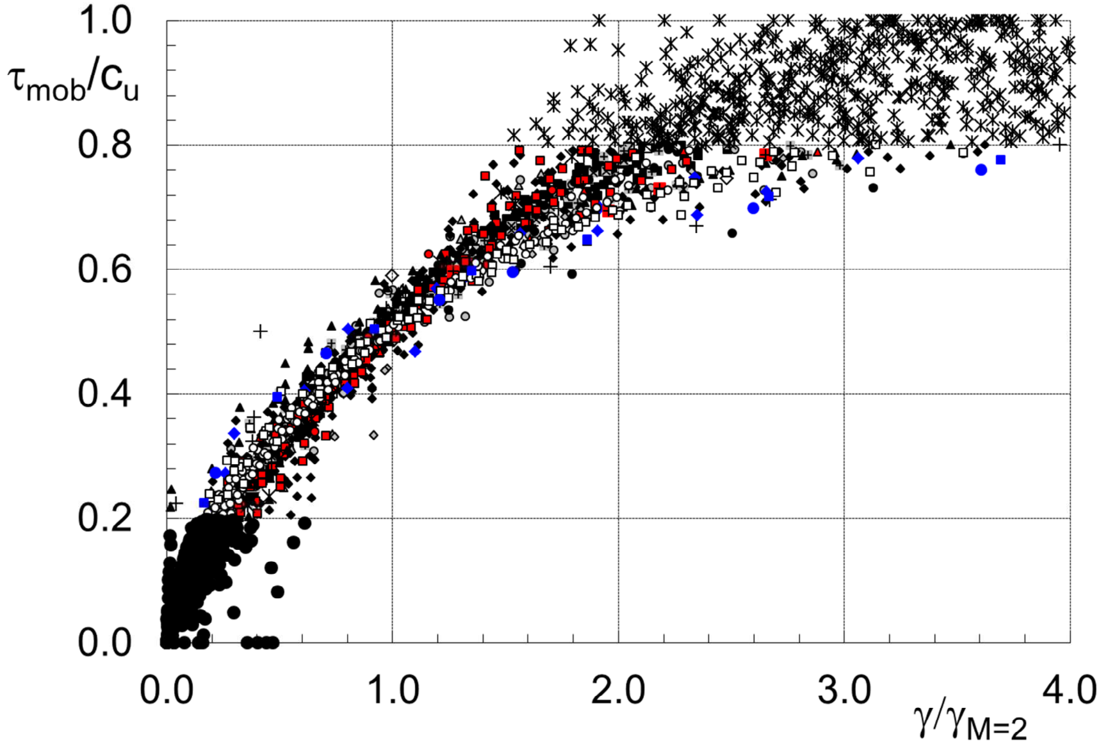

Figure 1 - Shear Stress Mobilization versus Normalised Shear Strain (Vardanega & Bolton,2011)

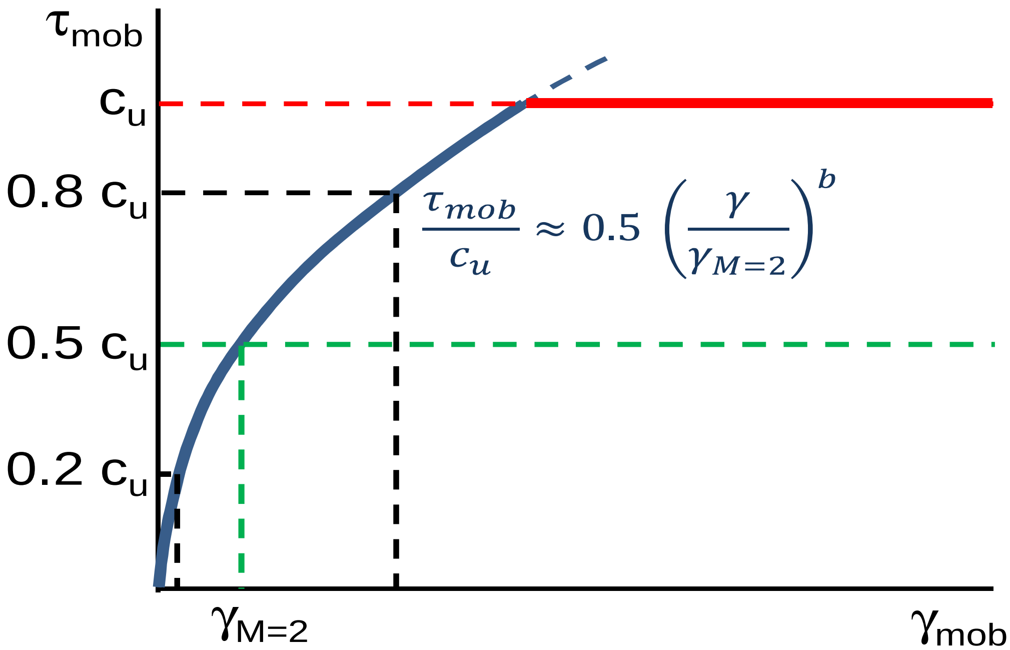

Figure 2 - The idealised mobilised strength curve using a power curve through the origin, intersected by a peak strength cap

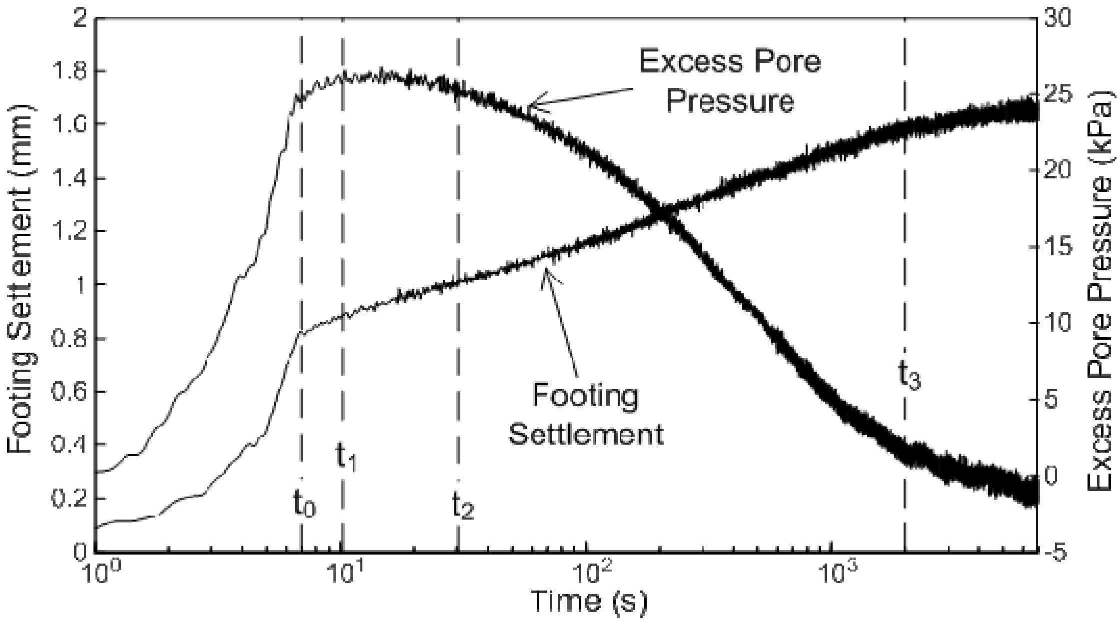

Figure 3 – Observed footing settlement versus time in centrifuge