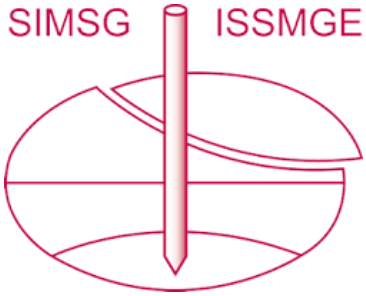
INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Shear strength of soft soil determined from element tests and penetration tests

Contrainte de cisaillement des sols mous déterminée à partir d'essais de laboratoire et de pénétromètre

H.A.Joer – *Centre for Offshore Foundation Systems, UWA*
M.Fahey – *Department of Civil and Resource Engineering, UWA*
M.F.Randolph – *Centre for Offshore Foundation Systems, UWA*
B.Bhattarai – *Centre for Offshore Foundation Systems, UWA*

ABSTRACT: The shear strength of soft soils such as clay or calcareous silt is an important parameter for the design of foundations in such sediments. In the field, the cone penetrometer test is the primary tool for estimating the shear strength, while in the laboratory, the strength is determined from element tests such as triaxial or simple shear tests. Correlations are then established between the results of the field and laboratory tests for design purposes. However, as is well known, the laboratory measurements may vary due to the type of test used (triaxial compression or extension, or simple shear) and the consolidation conditions (isotropic or anisotropic). The paper presents a comprehensive investigation of the shear strength of kaolin clay and a calcareous silt from the North West Shelf of Australia, involving element tests and model tests and comparisons are made between the results of the different tests.

RÉSUMÉ: La contrainte de cisaillement des sols mous, tel que l'argile ou la vase calcaire, est un paramètre important pour l'étude des fondations à installer sur ces sédiments. Sur le terrain, le pénétromètre est l'outil principal utilisé pour estimer la contrainte de cisaillement, alors qu'au laboratoire, la contrainte est déterminée à partir d'essais tel que le triaxial et le cisaillement simple. Les corrélations entre les résultats obtenus sur le terrain et ceux obtenus au laboratoire sont alors établies pour des fins d'avant-projets. Cependant, comme celà est bien reconnu, les mesures de laboratoire peuvent varier dû au type d'essai (triaxial de compression ou d'extension, ou cisaillement simple) et aux conditions de consolidation (isotrope ou anisotrope). L'article présente une étude compréhensive de la contrainte de cisaillement de l'argile kaolin et d'une vase calcaire de la côte ouest de l'Australie, incluent des essais de laboratoire et des essais sur modèles et une comparaison entre les résultats des deux types d'essais est effectuée.

1 INTRODUCTION

The shear strength of soft soils such as clay or calcareous silt is an important parameter for foundation design. In the field, the cone penetrometer test is the primary tool for estimating the shear strength, while in the laboratory, the strength is determined from element tests such as triaxial or simple shear tests. Correlations are then established between the results of the field and laboratory tests for design purposes. However, as is well known, the shear strength measured in the laboratory depends strongly on the type of test (triaxial compression or extension, or simple shear) and the consolidation conditions (isotropic or anisotropic). Inevitably therefore, correlations with in situ tests will also vary.

The paper presents a comprehensive investigation of the shear strength (s_u) of kaolin clay and calcareous silt from the North West Shelf of Australia, involving both triaxial and simple shear testing. A comparative analysis of the results obtained from these two types of test is presented. Block samples were prepared in a large container, 400 mm in diameter and 400 mm high. The soil was consolidated under a vertical pressure of 100 kPa. At the end of the consolidation phase, a model penetrometer with three different types of end shapes (cone, T-bar or ball) was pushed into the material and the bearing resistance was measured. Core samples of 72 mm in diameter were then taken from the large sample and isotropically and anisotropically consolidated triaxial and simple shear tests were carried out under confining stress varying from 25 to 250 kPa (OCR up to 4). Correlations between s_u and OCR are proposed for both materials and compared with ones previously published.

2 SAMPLE PREPARATION

The materials used in this study are a commercially available kaolin clay and a calcareous silt from the North West Shelf of Australia. The kaolin used has a liquid limit of 60, a plastic limit of 27 and a coefficient of consolidation (c_v) of 2 m^2/year. The calcareous silt is a well-graded material with a D_{50} of about 0.045 mm and a c_v of 60 m^2/year.

The preparation of the sample is the same for both materials. The dry material is first mixed with the required quantity of water (120 % for the kaolin clay and 42 % for the calcareous silt). The mixture is then poured into a container of 400 mm diameter and height of 800 mm. A vertical pressure (σ_{vp}) of 100 kPa is then applied on top of the sample which is left to consolidate. The consolidation is monitored and is terminated when no or little displacement is recorded. The final height of the trimmed sample is about 300 mm.

At this stage, the sample for the penetrometer tests in the large container is ready. After completion of these tests, the vertical pressure is then released and steel tubes of 72 mm diameter are pushed into the sample and taken for the element testing.

3 ELEMENT TESTING

Triaxial (TX) and simple shear (SS) tests are the most common element tests for deducing the undrained shear strength of soils in the laboratory. The principle of the triaxial test is widely published and is not discussed here. However the simple shear is not as common as the triaxial test and the apparatus used is briefly described here.

The simple shear apparatus used in this study was developed at UWA and allows consolidating a sample similarly to the triaxial setup, with the possibility of applying a back pressure and measuring the pore pressure. Following consolidation, the sample is then sheared horizontally. The boundary conditions used in this study are maintaining both the height of the sample and the total vertical pressure (σ_v) constant during shearing. The height of the sample is maintained constant by locking the vertical movement of the piston and σ_v is maintained constant by increasing or decreasing the cell pressure (σ_h) as required. Note that this latter condition is somewhat arbitrary, and does not af-

fect the effective stress path. However, it simulates a likely boundary condition in the field, and allows meaningful interpretation of the excess pore pressure ratio, $\Delta u/\sigma_{vc}$.

In this setup, shear forces, S, are applied on the upper and lower horizontal faces of the sample although the distribution of shear stress within the sample will be non-uniform, owing to the lack of complementary shear stress on the external vertical surface. However, assuming that the average shear stress within the sample may be taken as $\tau_{xy} = S/A$, where A is the cross-sectional area of the sample, and that the horizontal stress (σ_h) within the sample is equal to the cell pressure applied at the boundary, the principal effective stresses (σ'_1 and σ'_3) can be calculated as:

$$\sigma'_1 = \frac{\sigma'_v + \sigma'_h}{2} + \sqrt{\left(\frac{\sigma'_v - \sigma'_h}{2}\right)^2 + \left(\tau_{xy}\right)^2}$$

$$\sigma'_3 = \frac{\sigma'_v + \sigma'_h}{2} - \sqrt{\left(\frac{\sigma'_v - \sigma'_h}{2}\right)^2 + \left(\tau_{xy}\right)^2} \qquad (1)$$

The mean plane effective stress (s') and plane strain deviator stress (t) can then be calculated as:

$$s' = \frac{\sigma'_1 + \sigma'_3}{2} = \frac{\sigma'_v + \sigma'_h}{2}$$

$$t = \frac{\sigma'_1 - \sigma'_3}{2} = \sqrt{\left(\frac{\sigma'_v - \sigma'_h}{2}\right)^2 + \left(\tau_{xy}\right)^2} \qquad (2)$$

It should be noted that the assumption that σ_h is equal to the applied cell pressure is not exactly true, due to the low aspect ratio (typically 1 to 4) of the simple shear sample (Budhu, 1979).

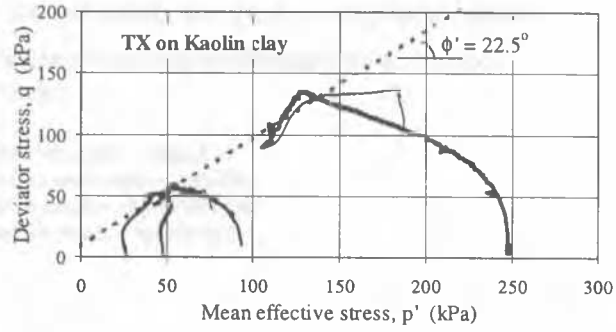
Both the TX and SS tests were carried out at confining pressures of 25, 50, 100, 200 and 250 kPa. For the tests carried out at OCR greater than 1, the consolidation is carried out in two stages. The first stage consists of consolidating the sample to a vertical pressure equal to σ_{vp}, in order to minimise the effect of stress relaxation due to coring and storage of the sample prior to testing. The second stage consists of reducing the confining pressure to the required value. Anisotropic TX and SS tests were carried out on the kaolin clay using a value for K_o of 0.6 (deduced from a "true-K_o" consolidation triaxial test) and only isotropic tests were carried out on the calcareous silt.

Figure 1 shows typical plots of deviator stress (q) versus mean effective stress (p') obtained from TX tests carried out on kaolin clay and calcareous silt. Figure 2 shows the corresponding plot (t versus s') obtained from SS tests on both materials. Linear trend lines are shown on the figures and the deduced values of friction angles are reported on the corresponding figure. For the kaolin clay a cohesion intercept of about 9 kPa for both the SS and TX is evident if a linear strength envelope is adopted through the test data.

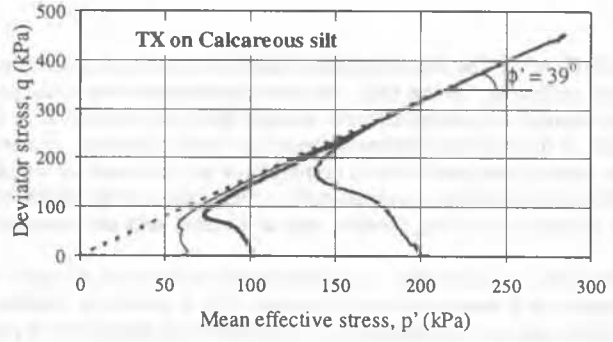
The element tests are compared using the approach proposed by Mayne (1985), plotting the normalised shear strengths, $(s_u/\sigma'_v)_{SS}$ and $(s_u/\sigma'_v)_{TX}$, from the two devices against each other (see Figure 3). Mayne (1985) proposed the relationships shown in the figure between these two parameters, suggesting that the undrained shear strengths determined from simple shear tests lie between 90 % and 50 % of those from triaxial tests. The experimental results presented here tend towards the upper bound suggesting that the difference between the two types of apparatus is narrower than previously reported. This may be due to the performance of the devices rather than the material properties.

Figure 4 shows the plot of s_u normalised by the initial confining effective vertical pressure (σ'_{vo}) versus the overconsolidated ratio (OCR). Upper and lower bound trend lines are also shown in the figure. These are consistent with previous work carried out on kaolin clay (Springman, 1989; Stewart, 1992).

Absolute values of s_u estimated for the samples subjected to an overburden pressure of 100 kPa are shown in the upper rows of Table 1, for comparison with results from the model penetration tests.

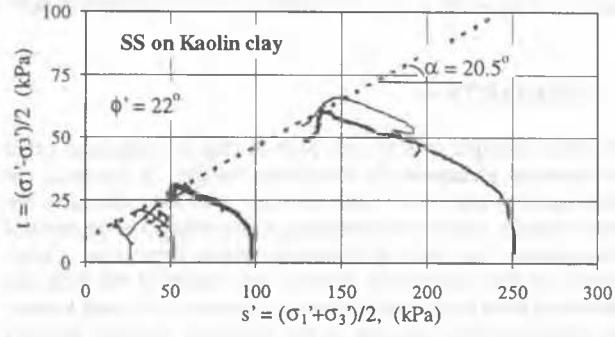


(a) Kaolin clay

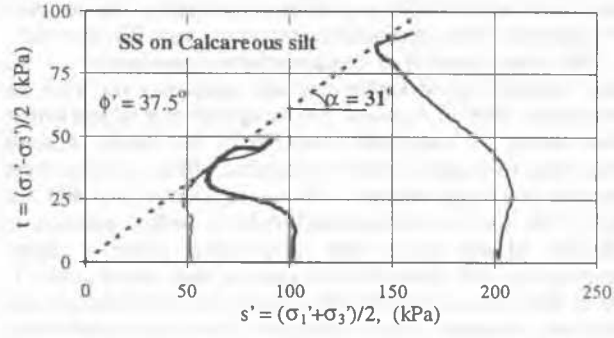


(b) Calcareous silt

Figure 1. Triaxial test results.



(a) Kaolin clay



(b) Calcareous silt

Figure 2. Simple shear test results.

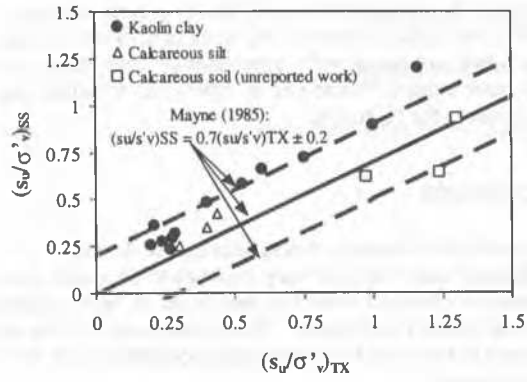


Figure 3. Comparison between triaxial and simple shear strength.

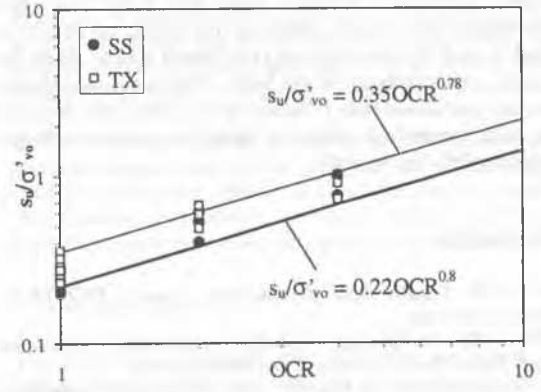


Figure 4. variation of su with OCR.

Table 1. Summary of test results.

		Soil	
		Kaolin clay	Calcareous silt
Shear strength, su (kPa)	TX	37	53
	SS	35	45
Resistance (kPa)	CPT	450	690
	TPT	360	610
	BPT	360	610
Bearing factors determined from TX results	N_c	12.2	13
	N_t	9.7	11.5
	N_b	9.7	11.5
Bearing factors determined from SS results	N_c	12.5	15.3
	N_t	10.3	13.5
	N_b	10.3	13.5

4 MODEL TESTING

Three different types of penetrometer devices were used in this study: the cone penetrometer (CPT), the T-Bar penetrometer (TPT) and the ball penetrometer (BPT). The device consists of a main shaft of 6 mm in diameter (D), with a removable tip in order to accommodate any of the three types used. This also allowed using the same load cell for all three devices, which is an advantage for eliminating eventual source of errors.

The CPT is a well-known device used extensively *in situ* for determining the soil profile and strength. The shear strength determined using the CPT is dependant on the corrections made to the output from the testing. As a matter of fact, corrections for the pore pressure at the shoulder of the cone and the estimation of the excess pore pressure due to the penetration of the cone are necessary. In addition, corrections due to the overburden stress are also necessary for determining the net cone resistance (q_{cnet}) from the measured cone resistance (q_{cm}). The following equation (Robertson and Campanella, 1983) shows the relationship between the corrected cone resistance and the measured value:

$$q_{cnet} = \frac{q_{cm} - \left(\sigma'_v + u_o\alpha\right)}{1-\left(1-\alpha\right)B_q} \tag{3}$$

where: σ'_v is the vertical effective stress, u_o is the ambient pore pressure, α is the pore pressure area correction and B_q is the ratio between the excess pore pressure and the net bearing pressure.

This equation shows how the estimation of the strength of the material is sensitive to the various parameters, which are often only estimated based on experience. In this study, the value of α has been determined as 0.9, using the method described by Lunne et al (1997).

The tests were carried out with a rate of penetration (v) of 0.017 mm/s. Finnie (1993) has shown that foundation loading may be regarded as undrained if the quantity vD/c_v exceeds about 10 and drained if it is less than 0.01. In this study, the quantity vD/c_v is about 1.6 for the kaolin clay and 0.05 for the calcareous silt. This suggests that the tests were carried out in a partially drained condition and therefore B_q has been taken as zero Hence only the correction due to the overburden pressure is taken into account, and since u_o is zero:

$$q_{cnet} = q_{cm} - \sigma_{vo} \tag{4}$$

The undrained shear strength (s_u) is then deduced from the CPT using the following equation:

$$s_u = \frac{q_{cnet}}{N_c} \tag{5}$$

where N_c is a cone factor determined from numerical and experimental work.

The TPT is a new type of penetrometer device developed at UWA (Stewart and Randolph, 1994). The principle is the same as for the CPT, with the tip being replaced by a cylindrical rod. In this study, the rod used is 6 mm in diameter and 24 mm in length. The advantage of this device compared to the cone is that the overburden stress and pore pressure act on both the top and bottom of the rod. Therefore the corrections made with the CPT are not required using this device, apart from over the connection between the bar and the load cell. In this study, this area was about 20 % of the horizontal projected area of the T-bar, and the small correction was neglected. The undrained shear strength (s_u) is then deduced from the TPT as follows:

$$s_u = \frac{q_t}{N_t} \tag{6}$$

where N_t is a T-Bar factor determined from analytical solutions and experimental studies (Randolph and Houlsby, 1984; Stewart and Randolph, 1991).

The BPT is similar to the TPT, with the tip being replaced by a ball of 10 mm in diameter. The ambient pore pressure and overburden stresses are also in equilibrium as they act all around the ball (neglecting the area for the connection between the ball and the shaft, which correspond to about 36 % of the total area). The flow around the ball is axisymmetric, compared with the plane strain flow assumed around the T-bar. The undrained shear strength is obtained using the following equation:

$$s_u = \frac{q_b}{N_b} \tag{7}$$

where N_b is a ball factor.

Figure 5 shows plots of bearing resistance (q_{cnet}, q_t and q_b) ver-

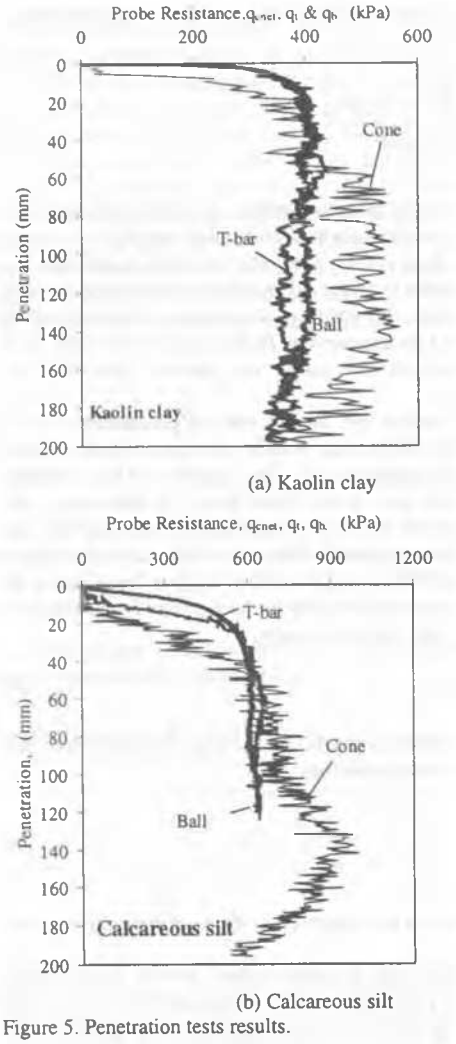


Figure 5. Penetration tests results.

sus penetration for the three types of penetration devices, obtained from the kaolin clay and calcareous silt samples prepared with applied overburden pressure (σ_{vp}) of 100 kPa. It can be seen that the CPT showed relatively larger resistance than the TPT and BPT tests although the measurements fluctuated more than those of the TPT and BPT. This is mainly due to the resolution of the load cell, as the projected area of the cone is 20 % and 36 % of those for the T-Bar and ball respectively. The results from TPT and BPT are very similar. The higher values of net cone resistance suggest that the value of N_c is greater than N_t and N_b and that N_b and N_t are practically the same.

Using the values of s_u obtained from the element tests along with Equations 4, 5 and 6, the values deduced for N_c, N_t and N_b are shown in Table 1. The absolute values of these parameters should be viewed with some caution, given the partially drained nature of the penetration tests. In practice, the values for kaolin clay are similar to other work (Watson et al. 1998), but the values for the calcareous silt, where the non-dimensional penetration velocity, vD/c_v, was only 0.5, are significantly higher.

Robertson and Campanella (1983) proposed values of N_c typically ranging from 7 to 15, depending on the sensitivity and degree of overconsolidation of the clay. In this study, average values for N_c of 14 (calcareous silt) and 12.5 (kaolin clay) are determined. Average values for N_t of 12.5 and about 10.5 were deduced for the calcareous silt and kaolin clay respectively. As mentioned earlier, the values of N_b are similar to those deduced for N_t. From the analytical work (Randolph and Houlsby, 1984), values of N_t ranging from 9.1 to 11.9 were proposed, depending on the interface roughness of the T-bar. Stewart and Randolph (1994) suggested a value of N_t of 10.5. Randolph et al. (2000) derived theoretical values of N_b ranging from 11.3 to 15.2, from upper and lower bound approaches and finite element analysis.

However, the higher value of N_b compared with N_t from theoretical work contrasts with experimental data from this study, and other studies (Watson et al. 1998), all of which suggest similar values for N_t and N_b.

5 CONCLUSIONS

From this study, the following conclusions can be drawn:

- The element tests showed very consistent data and good agreement is obtained from the two types of tests (simple shear and triaxial) performed. The friction angle of the silt was found to be about 39° under triaxial conditions and 37.5° in simple shear.
- The response from the penetration tests shows similar trends with available data in the literature.
- The bearing factors deduced from the T-Bar and ball penetrometers are similar, although the mode of failure in the soil around the two devices is different (plane strain for the T-Bar, axisymmetric for the ball). This is consistent with previously published data (Watson et al, 1998), but does not agree with theoretical solutions based on simple isotropic strength models for the soil.

6 REFERENCES

Budhu, M. 1979. Simple shear deformation of sand. *PhD Thesis, Cambridge University.*

Finnie, I.M.S. 1993. Performance of shallow foundations in calcareous soil. *PhD Thesis, The University of Western Australia*

Lunne, T., Robertson, P.K. & Powell, J.J.M. 1997. Cone penetrometer testing in geotechnical practice. *Blackie Academic and Professional, London.*

Mayne, P.W. 1985. A review of undrained shear strength in direct simple shear. *Soils and Foundations* : Vol. 25, No.3, 64-72.

Randolph M.F. & Houlsby G.T. 1984. The limiting pressure on a circular pile loaded laterally in cohesive soil. *Geotechnique, 34(4), 613-623.*

Randolph M.F. Martin C.M. & Hu Y. 2000. Limiting resistance of a spherical penetrometer in cohesive material. *Geotechnique, 50(5), 573-582.*

Robertson P.K. & Campanella R.G. 1983. Interpretation of cone penetrometer tests. Part I: Sand, *Canadian Geotechnical Journal, Vol. 20, 718-733.*

Robertson P.K. & Campanella R.G. 1983. Interpretation of cone penetrometer tests. Part II: Clay, *Canadian Geotechnical Journal, Vol. 20, 734-745.*

Springman, S.M. 1989. Lateral loading of piles due to simulated embankment construction. *PhD Thesis, Engineering Department, Cambridge University.*

Stewart, D.P. & Randolph M.F. 1991. A new site investigation tool for the centrifuge. *Proc. Intern. Conf. on Modelling – Centrifuge 91, Boulder, Colorado, 531-538.*

Stewart, D.P. 1992. Lateral loading of piled bridge abutments due to embankment construction. *PhD Thesis, The University of Western Australia*

Stewart, D.P. & Randolph M.F. 1994. T-bar penetration testing in soft clay. *Jour. Geot. Eng. Div., ASCE, 120 (12), 2230-2235*

Watson, P.G., Newson, T.A. & Randolph M.F. 1998. Strength profiling in soft offshore soils. *Proc. Of the First Intern. Conf. on Site Characterization, ISC'98, Atlanta, Georgia, USA, 1389-1394.*

7 ACKNOWLEDGEMENTS

The work presented here forms part of the activities of the Special Research Centre for Offshore Foundation Systems, funded by the Australian Research Council's Research Centres Program. This support is gratefully acknowledged. The involvement and assistance of technicians and undergraduate students in the Department of Civil and Resource Engineering at UWA is gratefully acknowledged.