Geotechnical Engineering Journal of the SEAGS & AGSSEA **Vol. 44 No.1 March 2013 ISSN 0046-5828**

Effects of Consolidation and Specimen Disturbance on Strengths of Taipei Clays

Richard N. Hwang[1], Za-Chieh Moh[1] and I-Chou Hu[1]
[1]*Moh and Associates, Inc., Taipei, Taiwan*
E-mail: richardwang@maaconsultants.com

ABSTRACT: Presented herein are the results of a study supplementing the one carried out in the early 90's for investigating the characteristics of Taipei clays for the design and construction of the Taipei Metro. It has been found that the lowering of piezometric level in the Chingmei Formation in the 70's has drastically increased the shear strengths of the clays which can be estimated by using the SHANSEP equations, as a result of consolidation. Furthermore, the shear strengths obtained in the routine unconsolidated undrained shearing tests are far too low due to specimen disturbance.

1. INTRODUCTION

An advanced study was conducted by Geotechnical Engineering Specialty Consultant (GESC) engaged by the Department of Rapid Transit Systems (DORTS) of Taipei City Government in the design stage of the metro system, as a designated task (the Designated Task, hereinafter) to study the characteristics of Taipei clays (Chin et al., 1994; Chin 1997, Chin and Liu 1997; and Hu et al., 1996). It was conducted in collaboration with a research team from Massachusetts Institute of Technology (MIT). The SHANSEP (Stress History and Normalized Soil Engineering Properties) concept was adopted to normalize strengths of clays to effective overburden stresses with over-consolidation ratio (OCR) as a primary index representing the history of previous loading (Ladd and Foott 1974). This paper presents the results of a supplemental study and discusses the effects of the lowering of piezometric levels in the Chingmei formation on the undrained shear strengths of Taipei clays.

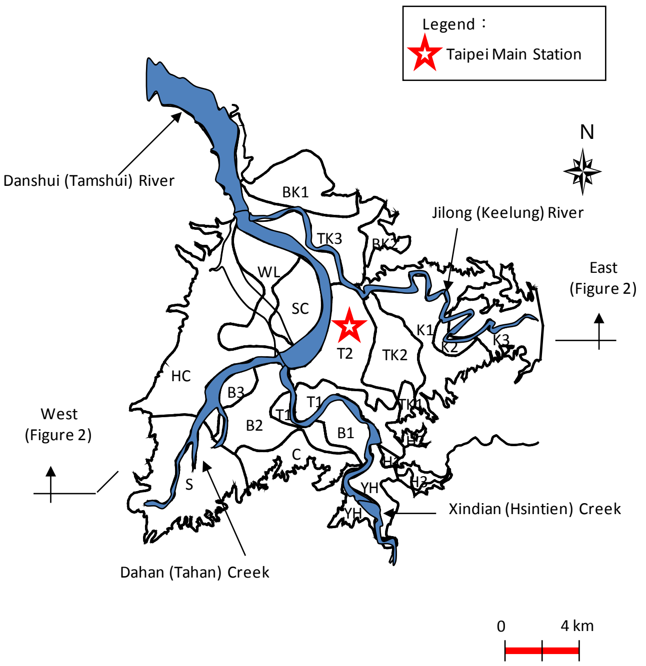

Figure 1 Geological map of the Taipei Basin (after Lee 1996)

2. GEOLOGY OF TAIPEI BASIN

Figure 1 shows a geological map of the Taipei Basin which was formed by deposition of sediments from the Tamshuei River and her three attributes, namely, the Keelung River, Hsientien Creek and Tahan Creek. Geological zoning was thus conducted accordingly and geological zones were designated as, e.g., T1 and T2 along the Tahan Creek, K1 and K2 along the Keelung River and H1 and H2 along the Hsientien Creek (MAA 1987; Chin et al. 2007; MAA Group 2007).

Figure 2 shows a soil profile along the Nangang-Banqiao Line of Taipei Metro. As can be noted, there exists at surface a thick layer of alluvial deposits, i.e., the so-called Sungshan Formation, which is underlain by the Chingmei Formation at depths varying from 40m to 70m. Figure 3 shows the results of a piezocone penetration test carried out at a location very near Taipei Main Station which, as depicted in Figure 1, is located near the center of the basin. The typical 6-layer sequence of subsoils in the Sungshan Formation is clearly identifiable from the profiles shown, particularly the one for pore pressure response. Layers I, III and V consist of mainly silty sands (Soil Type SM) and Layer II, IV and VI consist of mainly silty clays (Soil Type CL). As depicted in Figure 2, toward the west, the stratigraphy becomes more complicated with sandy and clayey seams frequently interbedded in the major layers; and toward the east, the sandy layers diminish and clays become dominating.

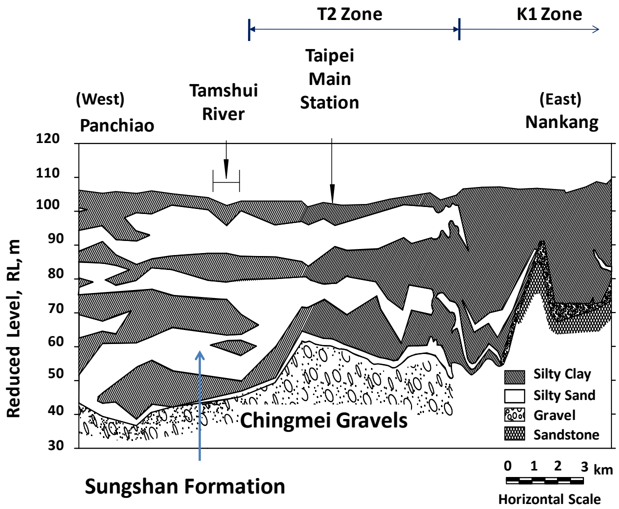

Figure 2 Soil Profile along the Nangang-Banquiao Line of Taipei Metro

3. CHANGES OF PIEZOMETRIC LEVELS IN CHINGMEI FORMATION

The Chingmei Formation is a very permeable water-rich gravelly aquifer and was once the sole source of water supply of the city. The piezometric level of groundwater in the Chingmei Formation was a few meters above the ground level at the turn of the 20th century (Wu 1968) and dropped drastically as a result of excessive pumping. It was closely monitored by Water Resources Planning Commission (WRPC) before the Commission merged into Water Resource Bureau (WRB) in 1996. Water Resource Bureau later merged into Water Resources Agency (WRA) in 2002 and the monitoring has been continuing ever since. Figure 4 shows the variation of piezometric level of groundwater in the Chingmei

Formation observed at North Gate which is about 0.5km to the west of Taipei Main Station. As can be noted, the piezometric level dropped to, as low as, EL-40m, which corresponds to a depth of 44m (ground level = EL+4m, or RL+104m). Because the Chingmei Formation is extremely permeable, the groundwater drawdown was widely spreading. Figure 5 shows the contour of piezometric levels in the Chingmei Formation in year 1975 (WRPC 1976). As can be noted, the drawdown was the largest in Xinzhuang and significant drawdown extended all the way to the rim of the basin.

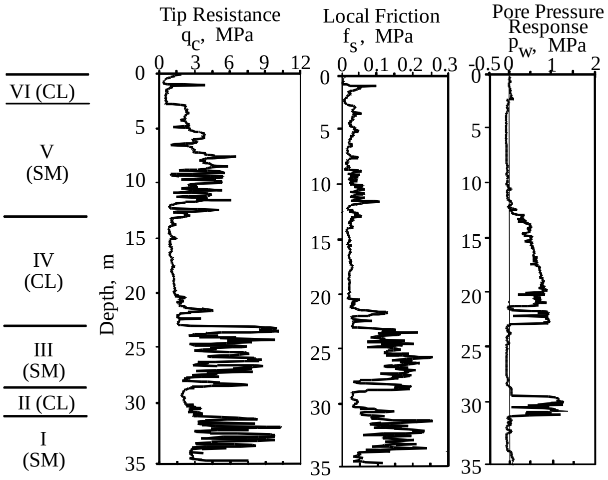

Figure 3 Results of piezocone penetration test in Sungshan Formation in central area of Taipei City

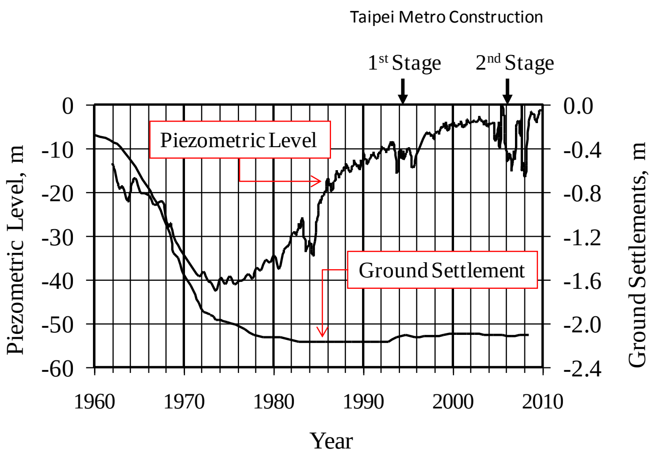

Figure 4 Piezometric level in the Chingmei Formation and surface settlements in central area of Taipei City

Alerted by the significant ground subsidence, the government started to regulate pumping of groundwater in 1971 and the piezometric levels became steady in the subsequent years. As depicted in Figures 4 and 6, the piezometric levels started to rise in the early 80's as surface runoff gradually replaced groundwater as the source of water supply. The recovery of piezometric levels spread over nearly the entire basin as can be noted by comparing Figure 7 with Figure 5. The recovery slowed down since the early 90's due to the lowering of groundwater pressures for maintaining the stability of deep excavations in several large infrastructure projects, particularly, the metro systems.

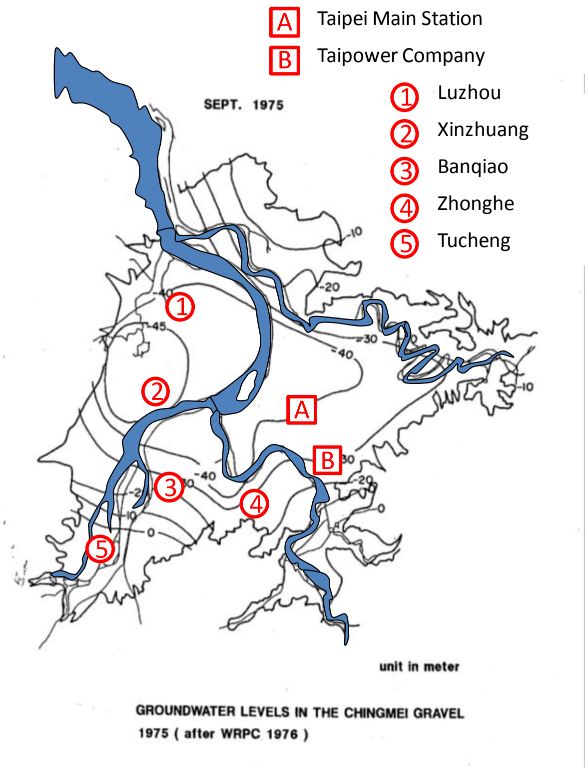

Figure 5 Contour of groundwater drawdown in the Chingmei Formation in 1975 (after WRPC 1976)

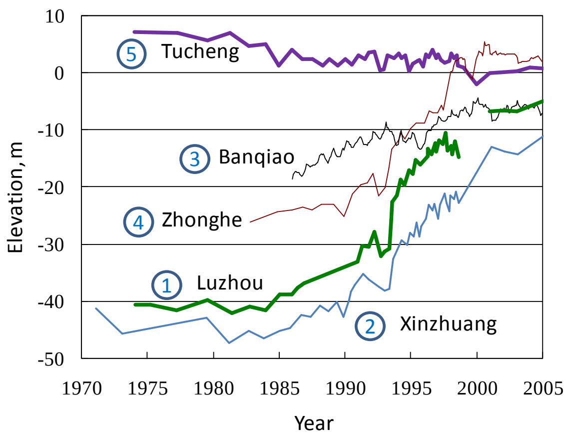

Figure 6 Changes of piezometric levels in the Chingmei Formation in suburbs

4. CONSOLIDATION OF SUBSOILS IN SUNGSHAN FORMATION

The lowering of piezometric level in the Chingmei Formation has led to significant reductions of water pressures in the overlying Sungshan Formation, and as a result, ground has settled by more than 2m in the central city area as shown in Figure 4. Based on the long-term records depicted in Figure 8, the piezometric level in Layer III in the central city area, where Taipei Main Station is, is estimated to be as low as EL-27m in the 70's. This level is about the same as the bottom level of Layer III, or even lower. In other words, the porewater pressure in Layer III was practically nil then. This is confirmed by Figure 9 which shows the changes of piezometric levels in various layers in the central city area in the past.

Figure 10 shows the changes in piezometric levels at the location of Taipower Company Building which, as depicted in Figures 1 and 5, is located toward the tip of T2 Zone near the rim of the basin (Woo and Moh 1990). Based on the data available in Figures 9 and 10, and with due consideration given to Figures 4 and 8, the changes in piezometric levels in Layer III at the locations of Taipei Main Station and Taiwan Power Company Building are estimated. The recovery curve for Taipei Main Station shown in Figure 11 is expected to be applicable to sites with piezometric levels in the Chingmei Formation dropping to a depth of 40m or lower and that for Taipower Company Building is expected to be applicable to sites with piezometric level dropping to a depth of 30m or lower.

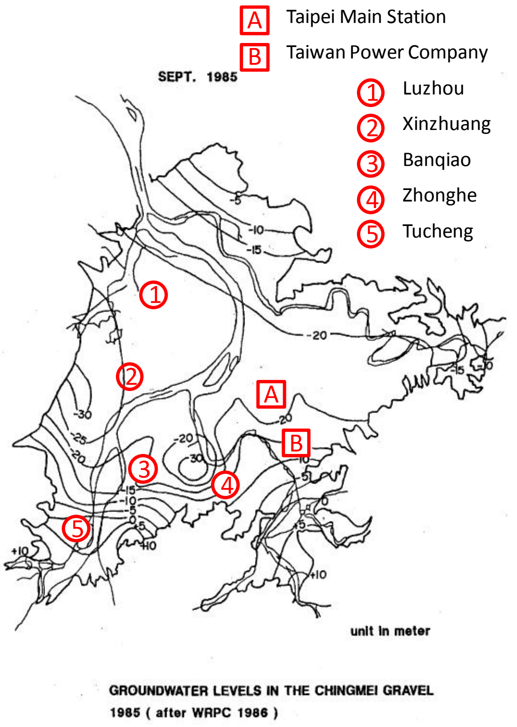

Figure 7 Contour of groundwater drawdown in the Chingmei Formation in 1985 (after WRPC 1986)

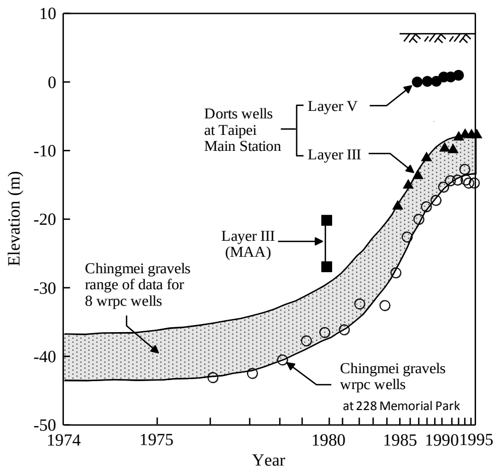

Figure 8 Drawdown of groundwater in the Sungshan Formation in the central area of Taipei City (after Chin 1997)

Regarding the piezometric level in Layer V, little information is available for estimating its lowest value. The underlying Layer IV is sufficiently thick to cut off the cross flow from Layer V to Layer III. It is thus reasonable to assume that the piezometric level in Layer V was unaffected by pumping of water from the Chingmei Formation. On the other hand, since the overlying Layer VI is thin, or even absent at places, Layer V is constantly recharged by surface runoff and the piezometric level in Layer V is primarily affected by the fluctuation of water levels in the rivers.

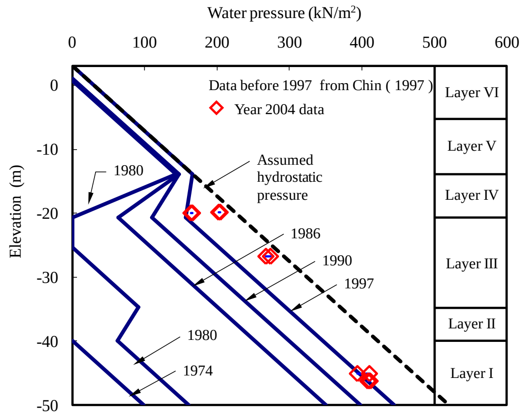

Figure 9 Changes in groundwater pressures in the Sungshan Formation in central area of Taipei City (after MAA Group 2007)

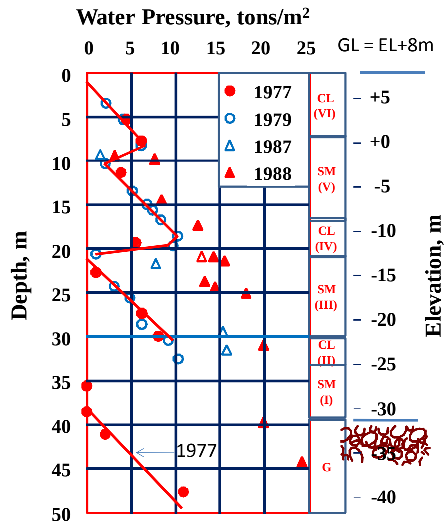

Figure 10 Drawdown of groundwater at Taipower Company Building (modified from Woo and Moh 1990)

An extensive program was conducted in 1979 and continued for 3 years to monitor the fluctuation of groundwater and the results indicated that the piezometric level in Layer V varied from EL+1.5 to EL-1.0m in the central city area (Ou el al. 1983). It has been noticed that pumping for lowering water pressures and for drawing water as a supply to construction activities during deep excavations in later years also affected the groundwater levels in shallow layers. But such influence was quite localized. For practical purposes, the piezometric level in Layer V can be assumed at a low of EL-1m and at a high of EL+2m.

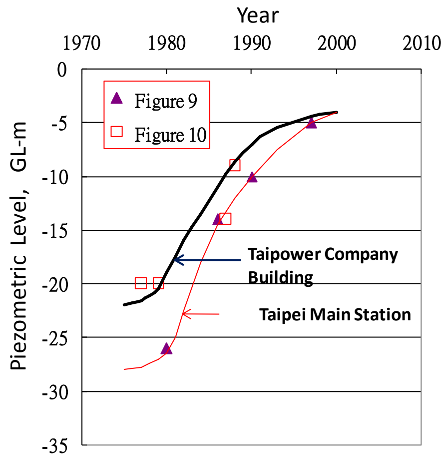

Figure 11 Changes in piezometric levels in Layer III of Sungshan Formation

Due to the reduction of porewater pressures, all the subsoils in the Sungshan Formation in the T2 Zone were substantially over-consolidated. This is particularly true for Layer II because the underlying Layer I is very permeable and the piezometric level in Sublayer I was practically the same as the piezometric level in the Chingmei Formation.

The time required for 99% consolidation can be estimated as follows (Duncan and Wright 2005):

$$t_{99} = 4D^2/C_v \qquad (1)$$

in which D = length of drainage path, C_v = coefficient of consolidation. The C_v values vary from, roughly, 1 cm^2/h to 100 cm^2/h for clays. The C_v values for silts are about 100 times the values for clays and the values for sands are about 100 times the values for silts. Accordingly, Figure 12 can be used to estimate the time required for the completion of consolidation in various types of subsoils of various thicknesses.

The so-called Taipei Clays, i.e., the clays in Layers II, IV and VI, are highly silty with more silts than clays. Therefore, consolidation would take less than 1000 days, or 3 years, for layers with drainage paths shorter than 10m to complete. Because the piezometric levels stayed at their lows for nearly 6 years, clay layers thinner than 20m would have been fully consolidated to the effective stresses corresponding to the piezometric levels observed in the 70's. It is thus anticipated that the consolidation was completed for the clay layers in the Sungshan Formation in the T2 Zone refer to in Figure 2.

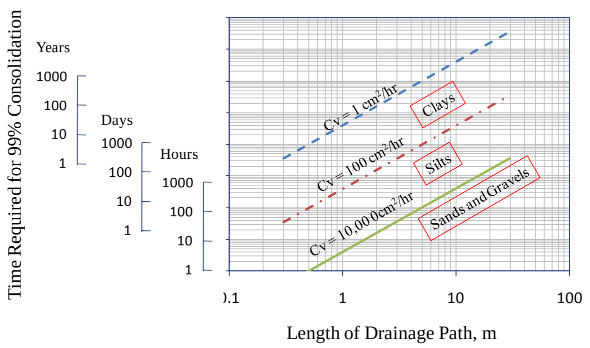

Figure 12 Time required for 99% consolidation (after Duncan and Wright 2005)

Figure 13 shows the stratigraphy of subsoils at Taipei Main Station, which is located near the center of T2 Zone as depicted in Figure 1, and Table 1 shows the estimated piezometric levels in Layers I, III and V in 1974, when the piezometric levels were at their lowest levels, and 1990 when the metro construction started. Table 2 compares the effective overburden pressures, σ_{vo}', in Layers II and IV in 1990 with those in 1974. In 1974, the piezometric level in Layer III was below the bottom level, so the porewater pressures, u, in this layer were practically nil. Therefore, the porewater pressures were nil at the bottom of Layer IV and at the top of Layer II. As depicted in Figure 11, the piezometric level in Layer III rose to EL-10m (or a depth of 14m) in 1990, giving a porewater pressure of 93 kPa at the bottom of Layer IV and 183 kPa at the top of Layer II. The effective stresses can thus be computed and an OCR value of 1.26 was obtained for the bottom-most portion of Layer IV and 1.41 for the top-most portion of Layer II. The effective overburden pressures at the top of Layer IV and bottom of Layer II can be computed in a similar way, giving an OCR value of 1.18 and 1.46, respectively as shown in Table 2.

The piezometric levels at the location of Taipower Company Building in 1974 and 1990 are also shown in Table 1. The effects of consolidation at this location were evaluated for the stratigraphy shown in Figure 10 and the results are given in Table 3. The OCR values obtained are more or less comparable with those obtained for Taipei Main Station.

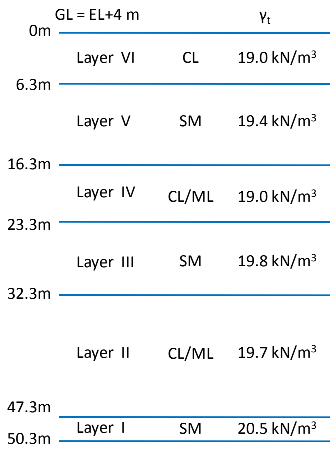

Figure 13 Subsoil stratigraphy at Taipei Main Station of Taipei Metro

The OCR values for Layer II and IV are thus expected to range, say, from 1.2 to 1.5, as depicted in Tables 2 and 3, for the entire T2 Zone in which the 6-layer sequence of subsoils is clearly identifiable and the clay layers are thin. They are also expected to be valid for clay layers with comparable thicknesses in other geological zones. For zones with clay layers thicker than 20m, for example, the K1 Zone, the degree of consolidation of clays demands a more thorough study.

A comprehensive research program, i.e the Designated Task, was conducted in 1991 by Geotechnical Engineering Specialty Consultant engaged by the Department of Rapid Transit Systems of the Taipei City Government in the beginning stage of the construction for Taipei Rapid Transit Systems (TRTS, or Taipei Metro) to study the engineering characteristics of subsoils in the Taipei Basin. It was conducted in collaboration with a research team from Massachusetts Institute of Technology (MIT) and a detailed investigation including drilling, sampling and laboratory tests was carried out. The testing program and results of tests are available in Chin et al. (1994), Chin (1997), Chin and Liu (1997), and Hu et al. (1996) and are well summarized in Chin et al. (2007) and MAA Group (2007).

Table 1 Piezometric levels in Sungshan Formation

Layer	Piezometric Level							
	Taipei Main Station				Taiwan Power Company Building			
	1974		1990		1974		1990	
	EL	GL	EL	GL	EL	GL	EL	GL
V	-1m	-5m	+2m	-2m	-1m	-9m	+2m	-6m
III	-34m	-38m	-10m	-14m	-20m	-28m	-4m	-12m
I	-40m	-44m	-12m	-16m	-30m	-38m	-10m	-18m

Table 2 Consolidation of subsoils in Layers II and IV at Taipei Main Station

Layer	Depth (m)	σ_{vo} (kPa)	1974		1990		OCR 1990
			u (kPa)	σ'_{vo} (kPa)	u (kPa)	σ'_{vo} (kPa)	
IV	16.3	314	113	201	143	171	1.18
	23.3	447	0	447	93	354	1.26
II	32.3	625	0	625	183	442	1.41
	47.3	920	33	887	313	607	1.46

Table 3 Consolidation of subsoils in Layers II and IV at Taipower Company Building

Layer	Depth (m)	σ_{vo} (kPa)	1974		1990		OCR 1990
			u (kPa)	σ'_{vo} (kPa)	u (kPa)	σ'_{vo} (kPa)	
IV	17	314	80	234	110	204	1.15
	21	389	0	389	90	299	1.30
II	30	563	20	543	180	383	1.42
	34	641	0	641	160	481	1.33

Shelby tube samples, 100mm in diameter and 762mm in length, were taken from three boreholes, i.e., R-1, R-2 and R-3, of which the locations are depicted in Figure 14. Borehole R-1 is located in the K1 Zone and Borehole R-2 in the T2 Zones. Borehole R-3 is located in a transition zone between T2 and K1 Zones and this transition zone was later defined, as depicted in Figure 1, as TK2 Zone in Lee (1996). Stationary piston sampling was carried out continuously to retrieve undisturbed samples. A section of the sample from Borehole R-1 was sent to Golder Associates, Calgary, Alberta, Canada for studying the state and yield surface and a portion of the sample taken from depth 18m to 22m was sent to MIT for special tests to study stress-strain-strength behavior (Germaine, 1992). All other tests were performed at MAA (Moh and Associates, Inc.) laboratory in Taipei.

Because the Taipei Clays exhibit rounded e-log stress curves, the "work/unit volume" approach proposed by Becker et al. (1987) was adopted to determine vertical yield stress, σ'_{vy} , in oedometer tests. The "work/unit volume" approach does not alter the data and the results obtained should be the same as the preconsolidation pressures determined by the conventional approach. Furthermore, the porewater pressures were those obtained from in-situ measurements. Despite the differences in approaches, the OCR values obtained are quite compatible with the inference based on the observed changes in piezometric levels as can be noted by comparing the values given in Tables 2 and 3 with those depicted in Figure 15. The agreement provides a mutual-check of the two sets of the data. It also implies that the subsoils in the Sungshan Formation had been fully consolidated to the pressures corresponding to the piezometric levels observed in the mid-70's.

For the clays at depths shallower than 15m, the OCR values and the Ko values are exceptionally large for reasons which are not readily clear. One of the speculations is that the large OCR and Ko values are the results of wave action during deposition in shallow water (Chin et al. 1994). Such a phenomenon was reported in Jefferies et al. (1987) for Arctic clays.

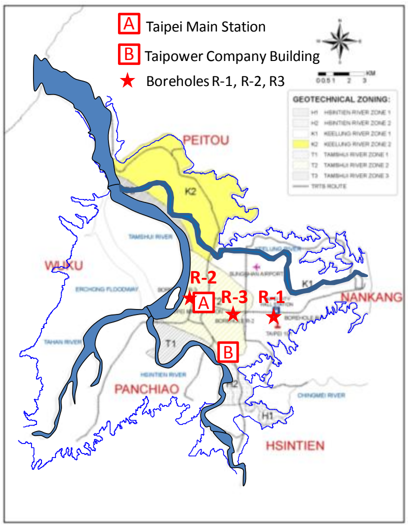

Figure 14 Locations of boreholes sunk in the Designated Task

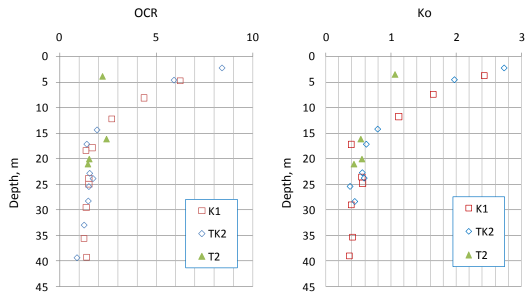

Figure 15 Over-consolidation ratios and Ko values obtained in the Designated Task

5. SHEAR STRENGTHS OF TAIPEI CLAYS

The undrained shearing strengths of clays obtained were normalized to the effective overburden pressures and are expressed by the so-called SHANSEP (Stress History and Normalized Soil Engineering Parameters) equation as follows (Ladd and Foott 1974):

$$S_u/\sigma'_{vo} = S \times OCR^m \qquad (2)$$

$$S = (S_u/\sigma'_{vo})_{nc} \qquad (3)$$

where, "nc" denotes normally consolidated and "m" is the linear slope of the expression of undrained strength ratio, i.e, S_u/σ'_{vo}, versus OCR on a log-log plot. Specimens were consolidated to various OCR values under the Ko conditions, followed by compression, extension and shearing tests. The normalized shear strengths obtained are shown in Figure 16 and, as can be noted, S = 0.32 and m = 0.82 were obtained in Ko-consolidated undrained triaxial compression tests (CKoUC). The corresponding S and m values are 0.19 and 0.82 for Ko-consolidated undrained triaxial extension tests (CKoUE) respectively, and 0.23 and 0.75 for Ko-consolidated direct simple shear tests (CKoDSS).

As can be noted from Figure 17, the plasticity indices, i.e., Ip values, for the Taipei clays vary from 5% to 25%. The S values obtained for normally consolidated clays, i.e., 0.32 for CKoUC, 0.19 for CKoUE and 0.23 for CKoDSS tests, are compatible with those reported in Ladd (1991) as shown in Figure 18.

Pairs of unsaturated-unconsolidated-undrained (UUU) tests and saturated-unconsolidated-undrained tests (SUU) were carried out on samples from Borehole R-1 (in K1 Zone) and Borehole R-3 (in TK2 Zone). The results are consistently lower than those obtained from CKoUC tests, as depicted in Figure 19. The difference could partly be attributed to specimen disturbance which will be discussed in Section 7. The effective stresses in specimens are unknown in the UUU tests because porewater pressures were not measured. The consolidation stress in SUU tests was only 5 kPa. That means, samples were consolidated under a confining pressure 5 kPa larger than the back pressure (MAA Group 2007). It is obvious that the actual effective stresses in the UUU and SUU tests are significantly lower than the in-situ stresses.

Tables 4 and 5 show the undrained shear strengths of soils in Layers II and IV at the locations of Taipei Main Station and Taipower Company Building estimated based on the effective stresses and OCR values given in Tables 2 and 3 and by using the SHANSEP equations given in Figure 16. The set of results for 1990 is in good agreement with those obtained from CKoUC tests as depicted in Figure 20 but are considerably greater than those obtained in the SUU and UUU tests. The shear strength at a depth of, for example, 40m is about 200 kPa while Figure 19 shows a value of about 80 kPa for SUU tests and 120 kPa for UUU tests. As can be noted from Tables 4 and 5, the shear strengths were reduced by, up to, 10% as a result of recovery of piezometric levels from 1974 to 1990.

The undrained shear strengths of clays at depths varying from 2m to 15m can be estimated by using the SHANSEP equation given in Figure 16. The data shown in Figure 15 suggests the following relationship:

$$OCR = 1.5 + 0.4(15-D) \qquad 2m<D<15m \qquad (4)$$

where, D = depth below surface. For groundwater table at a depth of 2m below surface and an average unit weight of 18.6 kN/m3, the undrained shear strengths will be 57 kPa at a depth of 2m and 66 kPa at a depth of 15m.

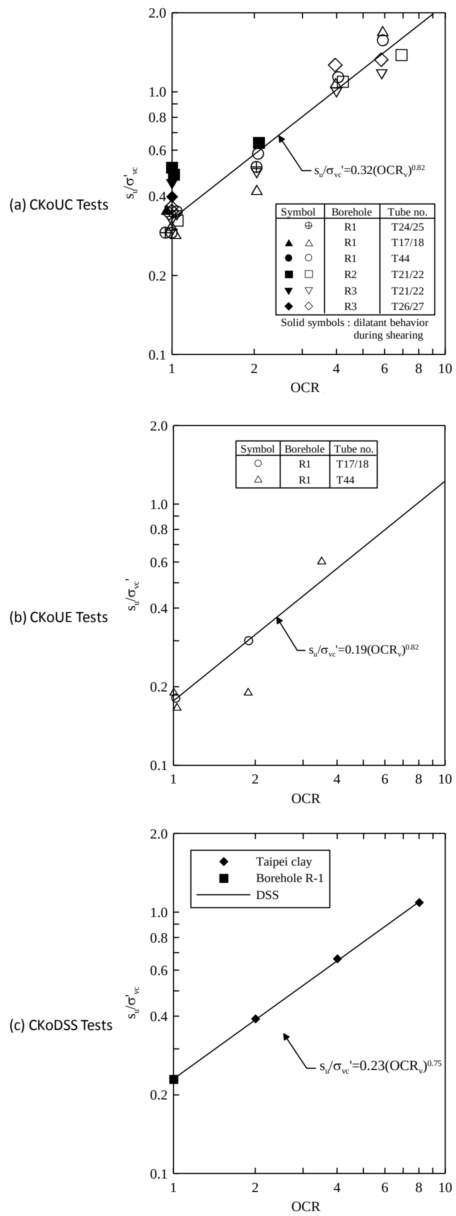

Figure 16 Undrained compression strength ratio versus OCR (after Chin et al. 1994)

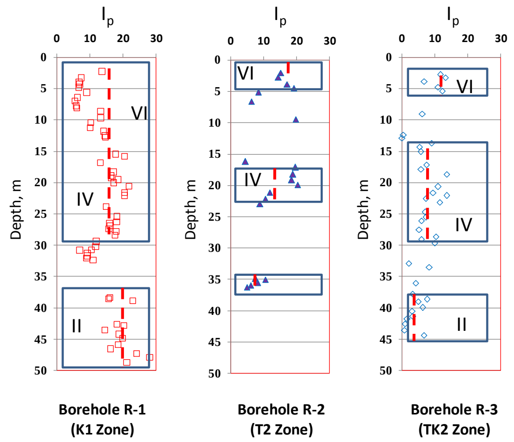

Figure 17 Plasticity indices for Taipei clays

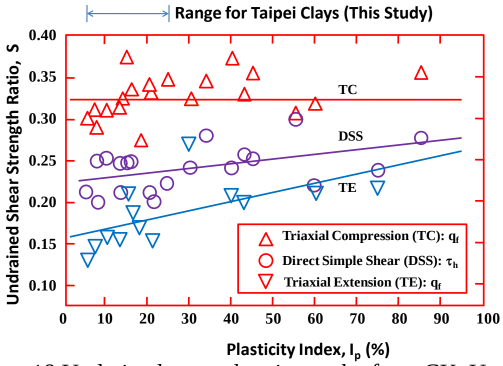

Figure 18 Undrained strength anisotrophy from CKoU tests on normally consolidated clays and silts (after Ladd 1991)

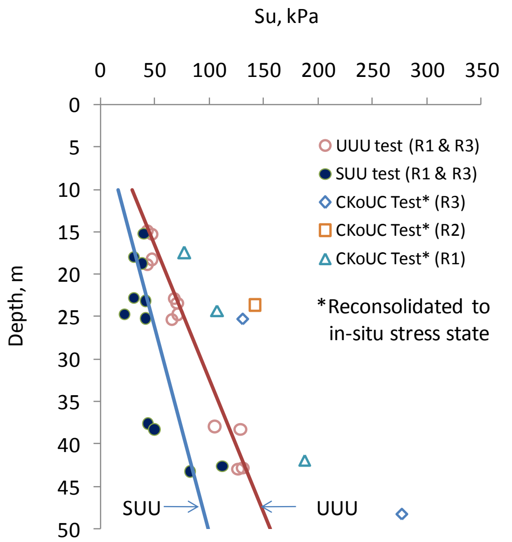

Figure 19 Comparison of undrained strengths from various tests (modified from Chin et al. 1994)

Table 4 Estimated undrained strengths of clays at Taipei Main Station

Layer	Depth	Estimated Undrained Shear Strength, kPa					
		Year 1974			Year 1990		
		CKoUC	CKoUE	DSS	CKoUC	CKoUE	DSS
VI	16.3m	38	22	27	36	21	25
	23.3m	117	69	84	111	66	78
II	32.3m	174	103	125	161	96	113
	47.3m	258	153	185	238	142	166

Table 5 Estimated undrained strengths of clays at Taipower Company Building

Layer	Depth	Estimated undrained Shear Strength, kPa					
		Year 1974			Year 1990		
		CKoUC	CKoUE	DSS	CKoUC	CKoUE	DSS
VI	17m	45	27	32	43	26	31
	21m	95	56	68	89	53	62
II	30m	144	86	104	133	79	93
	34m	175	104	126	165	98	116

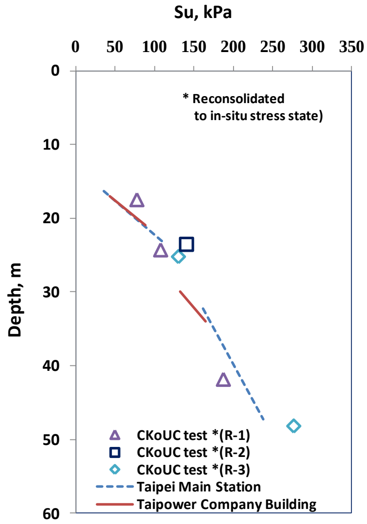

Figure 20 Comparison of estimated undrained shear strengths of clays at Taipei Main Station and Taipower Company Building with CKoUC test results

Figure 21 shows the profile for undrained shear strengths recommended to be used for practical purposes based on the results from CKoUC tests. Since the clays in the T2, TK2 and K1 Zones are quite similar, the figure is believed to be applicable to all these three zones.

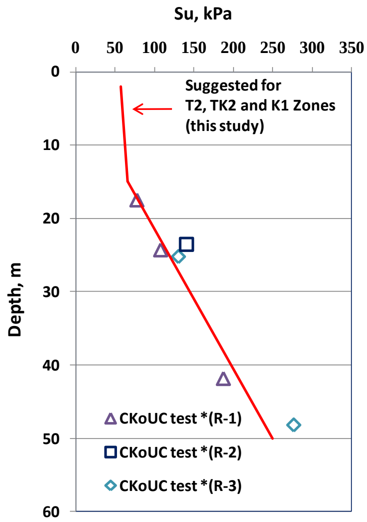

Figure 21 Estimated undrained shear strengths of clays in T2, TK2 and K1 Zones in 1990

6. ELASTIC MODULI OF TAIPEI CLAY

The normalized undrained secant moduli, E_u/σ'_{vo} , obtained in the Designated Task are plotted versus axial strain in Figure 22 (Chin and Liu 1997). The degradation of moduli with increasing axial strain is apparent. For CKoUC test results, it is found that the ratios of E_u/σ'_{vo} increase with increasing OCR at the same level of strain, ε_v. For CKoUE tests, the influence of OCR is not obvious, suggesting that the effect of previous overcosolidation is greater on vertical undrained modulus than on the horizontal one.

It is customary to normalize modulus of clays, E_u, to shear strengths, S_u, in engineering applications. Figure 23 shows the E_u/S_u ratios at strains corresponding to 50% ultimate strengths (Chin and Liu 1997). For CKoUC tests, a E_u/S_u value of 680 is obtained for normally consolidated clays, i.e., with OCR =1. It will be interesting to see how well this value compares with that proposed by others. Figure 24 shows the ranges of E values for clays with different plasticity indices (PI) proposed by Duncan and Buchignani (1976) based on back analyses of ground settlements. The 3 data points obtained by Chin and Liu (1997) for CKoUC tests fall on the lower bound for clays with PI<30. Notwithstanding the fact that these two sets of data were obtained based on different ratiocinations and, hence, direct comparison might not be meaningful, the trend that the E_u/S_u ratios decrease as OCR increases is nevertheless quite consistent. The relationship between E_u/S_u values and OCR proposed by Duncan and Buchignani (1976) can be normalized as follows:

$$(E_u/S_u)/(E_u/S_u)_{nc} = \exp(-0.09b) \tag{5}$$

$$b = (OCR-1)^{1.4} \tag{6}$$

This relationship is valid for the range of OCR of practical interest, say, OCR<6, and is expected to be applicable to shear strengths obtained in CKoUC tests on Taipei clays.

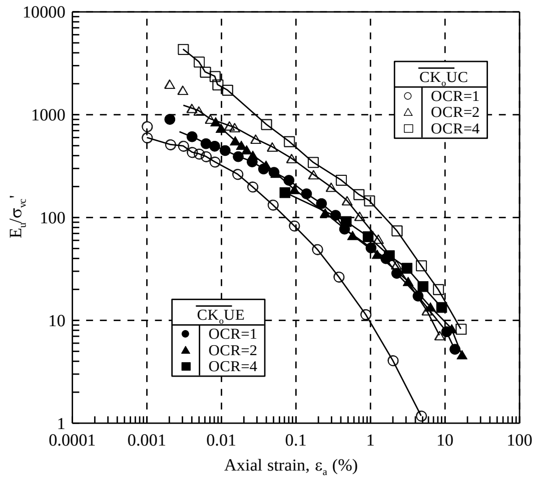

Figure 22 Normalized undrained modulus degradation for Taipei Clay (after Chin and Liu, 1997)

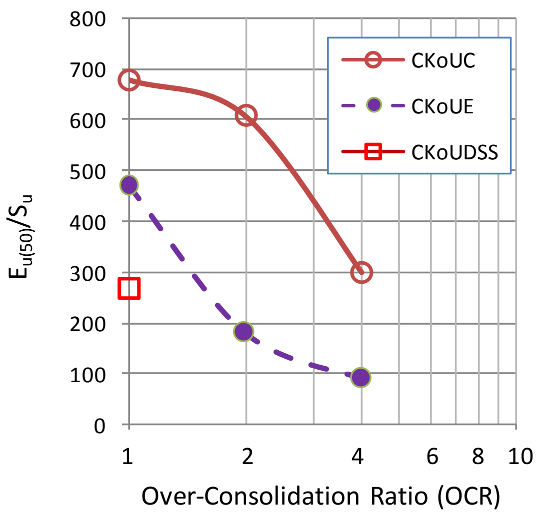

Figure 23 Undrained modulus of Taipei Clay (Chin and Liu, 1997)

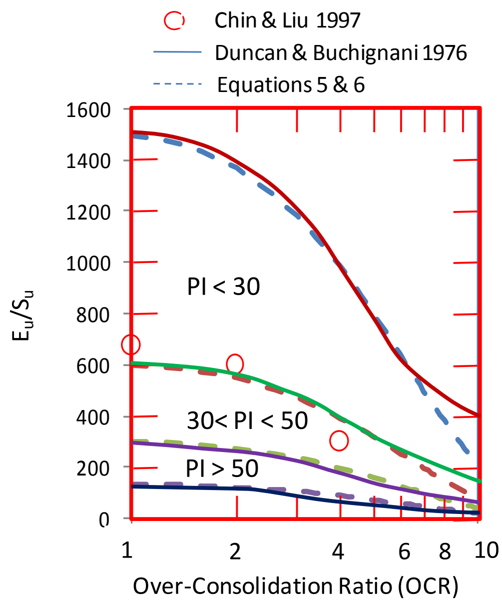

Figure 24. Influence of OCR on undrained modulus of clays

7. EFFECTS OF SPECIMEN DISTURBANCE

It is a well-known fact that the soil strengths obtained from laboratory tests are very much affected by specimen disturbance. Specimen disturbance may occur during sampling and handling, during transit to the laboratory, during laboratory storage and trimming of specimens for tests. The most serious mechanism of disturbance is shear distortion of the natural soil structure produced by displacement of the soil during conventional tube sampling and careless handling of the sample. Care must be taken to maintain sufficient hydro-pressures in boreholes to avoid swelling/failure of soils at the bottom. In the Designated Task, the unit weight of drilling mud was maintained at 1.05 t/m^3 or above for reducing disturbance to samples. A detailed discussion on the effects of disturbance in each of these stages is available in Ladd and DeGroot (2003).

Andresen and Kolstad (1979) proposed to adopt the axial strains subjected to the in-situ vertical stress in oedometer tests, or the in-situ effective vertical and lateral stresses in triaxial tests, as indices of the quality of specimens. The criterion for evaluating sample disturbance is presented in Table 6. Terzaghi et al. (1996) adopted this criterion, however, redefined the designations from A to E as depicted in the table. Specimens of A quality are difficult to obtain, specimens of B quality are adequate for practical purposes (Terzaghi et al. 1996).

Table 6 Classification of Specimen Quality

Volumetric Strains (%)	Specimen Quality Designation (SQD)	
	Andresen and Kolstad (1979)	Terzaghi et al. (1996)
<1	Very good to excellent	A
1-2	Good	B
2-4	Fair	C
4-8	Poor	D
>8	Very poor	E

It has been reported that the unconfined compression undrained shear strengths of specimens of D quality can be less than 50% of those of specimens of A quality (Terzaghi et al. 1996). Subjecting specimens of D quality to the field effective stress condition does not restore the natural structure of the soil, and the water content at the end of the consolidation stage is usually significantly less than the in-situ water content. Therefore, in general, D to C quality specimens consolidated to effective stresses below the preconsolidation pressure cannot provide information on the in-situ strength. On the other hand, if the samples are consolidated beyond the preconsolidation pressure, the undrained strengths can be estimated based on the S_u/σ'_{vo} ratios (Terzaghi et al., 1996).

Hu et al. (2004) adopted the approach proposed by Andresen and Kolstad (1979) and analyzed the quality of specimens made out of samples from 6 sites in the Taipei Basin, 1 in T2 Zone, 1 in TK2 Zone and the rest to the west of the Tamshui River, refer to Figure 1. A total of 26 consolidation tests were performed and the results are given in Figure 25. As can be noted, only the 3 specimens at shallow depths can be rated C quality and all other specimens are rated Class D or poorer. The volumetric strains for some of the samples even exceeded 12%. There is a tendency for specimen quality to get poorer as depth increases. Therefore, it is very doubtful that the strengths obtained from specimens of such quality are representative of in-situ strengths.

In contrast, all the specimens tested in the advanced study in Designated Task carried out by GESC were of B quality (Hu et al. 2004). The Designated Task was conducted as a research project under stringent supervision. Therefore the test results shown in Figures 15 through 23 are much more reliable than those obtained in usual projects. For example, as mentioned, an undrained shear strength of 200 kPa is given in Figure 21, while the Su values obtained from UUU tests is estimated to be about 120 kPa at a depth of 40m, refer to Figure 19, and a value of 80 kPa is often reported in design submissions.

UUU tests are widely used in practice throughout the world to obtain design values of Su for undrained stability analyses of loads on soft clay. It was pointed out in Ladd and DeGroot (2003) that such test often exhibit large scatter, especially with increasing depth and more fundamentally, reliance on UUC tests to estimate S_u depends on a fortuitous cancellation of three errors:

(a) The fast rate of shearing (60%/hr) causes an increase in the measured S_u;

(b) Shearing in triaxial compression also causes an increase in S_u since it ignores the effects of anisotropy;

(c) Sample disturbance causes a decrease in S_u.

These compensating errors cannot be controlled and only pure luck will yield a strength equal to S_u, i.e., such that disturbance offsets the higher strength due to fast shearing in triaxial compression. If one runs UUC tests on high quality samples, the Su values can be too high (unsafe) by more than 25 to 50% and the UUC strengths from low quality samples can easily be 25 to 50% too low.

It was further concluded by Ladd and DeGroot (2003) that UUC tests generally are a waste of time and money and have little advantage over less costly strength index tests like the Torvane, lab vane and fall cone; and the cost savings will be better spent on consolidation tests and Atterberg Limits, which can then be used with a quality C estimate of "S" and "m" values, refer to Eqs. 2 and 3, in order to directly calculate S_u or to check strengths estimated from in situ vane or piezocone tests.

In conventional analyses for, for example, slope stability the errors associated with the adoption of low shear strengths usually obtained in UUC tests were, hopefully, compensated by factors of safety which were empirically calibrated against observations. If higher soil strengths were adopted, specifications should be revised to accommodate the changes, otherwise, there could be risks of failure. Similarly, for deep excavations, active and passive earth pressures were traditionally computed on the assumption that soil strengths would be fully developed and the retaining systems were designed to resist the computed forces and bending moments with empirical factors of safety. Ground movements are assumed to be adequate if sufficient factors of safety are applied. The use of UUU strengths usually is therefore adequate in most cases.

However, it has become more and more popular to adopt numerical schemes, such as finite element and finite difference methods, nowadays to compute deformations of soil-structure interaction systems as well as the associated forces and bending moments in structural elements. For example, deflections of retaining walls in deep excavations and ground movements are routinely demanded to be computed so the potential of damage to adjacent structure can be assessed. In such cases, it is essential to use strength parameters corresponding to in-situ stresses for the results to be comparable with the reality. This is particularly true for new excavations in close proximity to existing metro tunnels of which the allowable deformations are very limited.

The process of Ko-consolidation in the SHANSEP approach would re-establish the in-situ stresses in soils and reduce specimen disturbance, and therefore, more consistent and more representative soil strengths will be obtained. However, since the shear strengths obtained would be much greater than the UUU strengths as evidenced in Figure 19, the possibility for designs to be insufficient can not be ruled out if these strengths are used. For short term loading, for example, deep excavations which are temporary works and usually take only a few months to complete, the use of higher strengths is deemed justifiable. As a principle, the adequacy of the use of SHANSEP approach in various types of analyses has to be verified by back analyses.

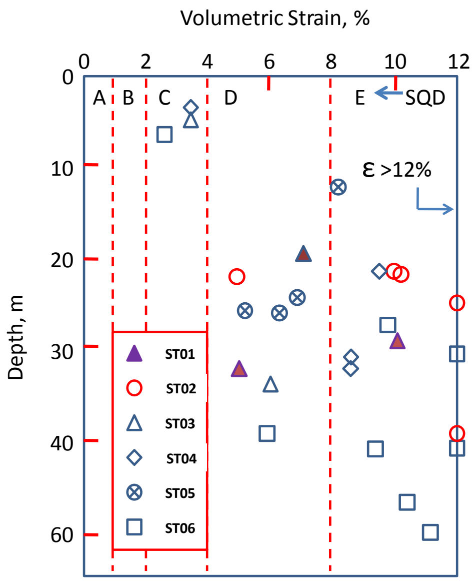

Figure 25 Quality designations for specimens reported in Hu et al., 2004

8. CONCLUDING REMARKS

Based on the fore-going discussions, it is concluded that:

1. Due to specimen disturbance, the shear strengths obtained from undrained shearing tests on unconsolidated specimens are scattering and are deemed unsuitable for analyses of which the purpose is primarily to obtain deformations of ground or deformations of soil-structural interaction systems;
2. Because of the difficulty in obtaining specimens of high quality, the strengths of clays obtained from laboratory tests are likely under-estimated, particularly for soils at great depths;
3. The strengths of clays can be estimated by using the SHANSEP equations with the consolidation effects due to the lowering of piezometric level in the Chingmei Formation properly accounted for;
4. The Ko-consolidation process in the SHANSEP approach will reduce the effects of specimen disturbance and, hence, the

strengths obtained will be more consistent and better representative of in-situ strengths;

5. The adequacy of the use of SHASEP approach in various types of analyses has to be verified by back analyses.

9. REFERENCES

Andresen, A. and Kolstad, P. (1979) The NGI 54mm samplers for undisturbed sampling of clays and representative sampling of coarser materials, Proceedings of International Symposium on Soil Sampling, Singapore, pp. 1-9

Becker, D. E., Crooks, J. H. A., Been, K. and Jefferies, M. C. (1987) Work as a criterion for determining in-situ and yield stresses in clays, Canadian Geotechnical J., v24, no. 4, pp 549-564

Chin, C. T. (1997) Panel Discussion: Groundwater control during the construction of Taipei MRT, Proc., 14th ICSMFE, Hamburg, Germany, September 6~12, pp.2343-2346

Chin, C. T. and Liu, C. C. (1997) Volumetric and undrained behaviors of Taipei silty clay, J. of Chinese Institute of Civil and Hydraulic Engineering, v9, no. 4, pp. 665-678, Taiwan (in Chinese)

Chin, C. T., Cheng, T. T. and Liu, C. J. (1989) Relationship between undrained shear strength and over-consolidation ratio of Taipei Silt, J. of the Chinese Inst. Of Civil and Hydraulic Engineering, v1, no. 3: 245-250 (in Chinese)

Chin, C. T., Crooks, J. H. A. and Moh, Z. C. (1994) Geotechnical properties of the cohesive Sungshan deposits, Taipei, J. Geotechnical Engineering, Southeast Asian Geotechnical Society, v25, no. 2, December, Bangkok, Thailand

Chin, C. T., Chen, J. R., Hu, I-C., Yao, D. T. C. and Chao, H-C. (2007) Engineering characteristics of Taipei Clay, in T.S. Tan, K. K. Phoon, D. W. Hight & S. Leroueil (Eds), entitled: Characterisation and Engineering Properties of Natural Soils, v3, pp. 1755-1803. © Taylor & Francis Group

Duncan J. M. and Buchignani, A. L. (1976). An engineering manual for settlement studies. Geotechnical Engineering Report, University of California, Berkeley

Duncan, J. M. and Wright, S. G. (2005) Soil strength and slope stability, John Wiley and Sons, Hoboken, New Jersey

Germaine, J. T. (1992) Test results from special laboratory testing program, Taipei Subway, Report to Moh and Associates, Inc., Taipei

Hu, I- C., Chin, C. T. and Liu, C. J. (1996) Review of the geotechnical characteristics of the soil deposits in Taipei, Sino-Geotechnics, 54:5-14 (in Chinese)

Hu, I-C., Chin, C. T. and Wu, S. H. (2004) Disturbance and Specimen Quality Designation of Cohesive Soil, Sino-Geotechnics, no. 103, pp. 83-88 (in Chinese)

Jefferies, M. G., Crooks, J. H. A., Becker, D. E. and Hill, P. R. (1987) Independence of Geostatic stress from over-consolidation in some Beaufort Sea Clays, Canadia Geotechnical J., v24, no. 3, pp 342-356

Ladd, C. C. (1991) Stability evaluation during staged construction, 22nd Terzaghi Lecture, J. of Geotechnical Engineering Division, ASCE, 117(4), 540-615

Ladd, C. C. and Foott, R. (1974) New design procedure for stability of soft clays, J. of Geotechnical Engineering Division, ASCE, 100 (GT7): 763-786

Ladd, C.C. and DeGroot, D.J. (2003) Invited Paper: "Recommended practice for soft ground site characterization." The Arthur Casagrande Lecture, Proceedings of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering, Boston, MA, 3-57.

Lee, S. H. (1996) Engineering geological zonation for the Taipei City, Sino-Geotechnics, 54, 25-34 (in Chinese)

MAA (1987) Engineering properties of the soil deposits in the Taipei Basin, Report No. 85043, Ret-Ser Engineering Agency and Taipei Public Works Department, Taipei (in Chinese)

MAA Group (2007) Engineering characteristics of Taipei Clay, MAA Group Consulting Engineers, Taipei, Taiwan

Ou, C. D., Li Y-G and Cheng, T-J (1983) The influence of distribution of ground water pressure on the foundation engineering in Taipei Basin, J. of the Chinese Institute of Civil and Hydraulic Engineering, vol. 10, no. 3, Nov. 1983, pp. 89-102 (in Chinese)

Terzaghi, K., Peck, R. B. and Mesri, G., Soil Mechanics in Engineering Practice, 3rd Ed. Wiley-Interscience (1996) ISBN 0-471-08658-4

Woo, S. M. and Moh, Z. C. (1990) Geotechnical characteristics of soils in the Taipei Basin, Proc., 10th Southeast Asian Geotechnical Conference, vol. 2, pp. 51-65, Taipei

Wu, C. M. (1968) Subsidence in Taipei Basin, Part II, J. of the Chinese Institute of Civil and Hydraulic Engineering, Taipei, Taiwan, v4, pp 53~81 (in Chinese)

WRPC (1976; 1986; 1987; 1988; 1989) Level survey of benchmark network of the Taipei Basin, Annual Report of Water and Resources Planning Commission (in Chinese)