EVALUATION OF JET GROUTING BY IN-SITU TESTS

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ABSTRACT

Various types of testings to assess the effectiveness of ground improved by jet grouting in clays were conducted. The undrained shear strengths and the elastic moduli of the grouted soil mass were evaluated by the unconfined compression tests conducted on the cored specimens, the cone penetration tests and by the pressuremeter tests. Although the undrained shear strengths interpreted by these 3 methods were within the same range, the average values obtained by the unconfined compression tests were slightly greater than that obtained by the pressuremeter tests. It was consistent with the fact that the laboratory tests were generally conducted on specimens of good quality while the in-situ tests did not discriminate against inferior zones of ground treatment. Results of the unconfined compression and the in-situ tests showed that the shear strengths increased with time. Correlations between the undrained shear strengths and the elastic moduli were established based on the results of the laboratory and the in-situ testings. An empirical cone factor for assessing the undrained shear strengths from the cone penetration tests was back-calculated from 2 sets of the pressuremeter tests.

1.0 INTRODUCTION

Jet grouting has been extensively used in recent years as a settlement mitigation measure around deep excavation sites in soft clay. By improving the strength and the stiffness of the subsoil, the settlements of the adjacent structures can be minimized. The strength and the stiffness of the improved ground are vital factors which require a proper assessment prior to the excavation.

On the other hand, there are various methods of jet grouting, such as the single-tube and the double-tube techniques. Even the same technique is adopted, grouting can be executed under various design parameters including the diameter and the spacing of grout piles, mix proportion, pressure, rate and amount of injection and so on. The acceptability and the quality control of ground treatment are of particular concern.

This paper presents the case histories of the provision of jet grouting mats at 3 underground stations of the Taipei Rapid Transit Systems (TRTS). Jet grouting was implemented by 4 different methods. The effectiveness of ground treatment was verified by laboratory testings on cored specimens, and also by cone penetration tests and the pressuremeter tests. The results of these testings provide valuable information on the treated soil.

2.0 SITE CONDITIONS

The sites for Stations BL 13, BL 14 and BL 15 are located on the eastern side of the Taipei Basin. The subsoil is dominated by a thick soft clayey layer with a thickness of about 40 m. The typical soil properties are presented in Fig. 1. The groundwater level is located at a depth of about 3.5 m.

3.0 GROUND TREATMENT

The 3 underground stations, BL 13, BL 14 and BL 15, of the Nankang Line, TRTS, are located in close proximity to existing buildings. The buildings are typically 4-storey reinforced concrete frame structures supported on isolated footings which are founded in soft clays. The depths of excavation range from 16.7 m to 20.4 m. Although diaphragm walls and internal bracing systems would be installed as excavation supports, it was anticipated that maximum lateral deformations of the diaphragm wall ranging from 80 mm to 120 mm and that maximum ground settlements ranging from 60 mm to 80 mm might occur.

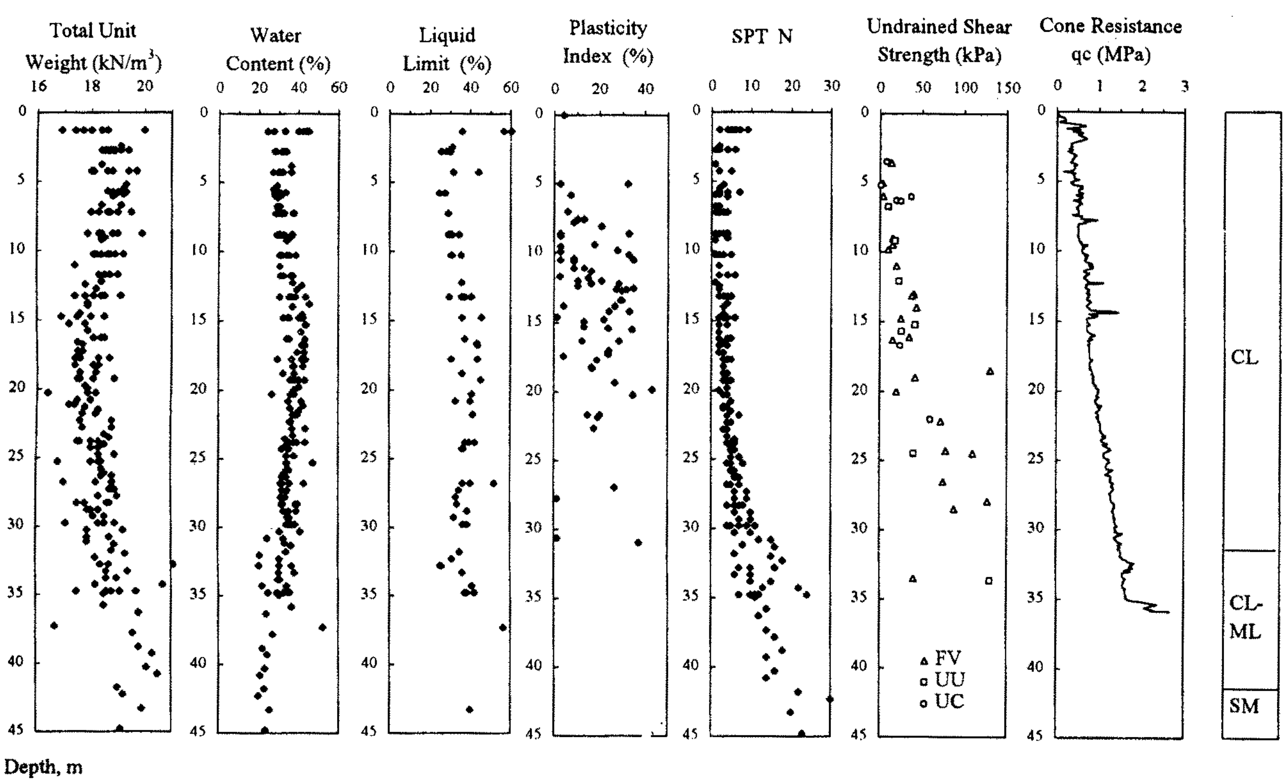

Fig. 1 : Typical soil properities at Stations BL13 to BL15

Table 1: Extent of the ground treatment

Station	Extent of Grouting			Grouting Method
	Length m	Width m	Depth m	
BL 13	276	25	18.8-22.8	JSG JG
BL 14	250	20	16.7-19.7	SWING UHPG
BL 15	60	20	20.4-24.4	UHPG

In order to minimise the settlements of the adjacent buildings, a grouted mat was provided prior to excavation in each of the station boxes. The thicknesses of the grouted mats vary from 3 m to 4 m. Table 1 summarizes the extent of the grouted mats for the 3 stations and Fig. 2 shows the typical arrangement of the ground treatment.

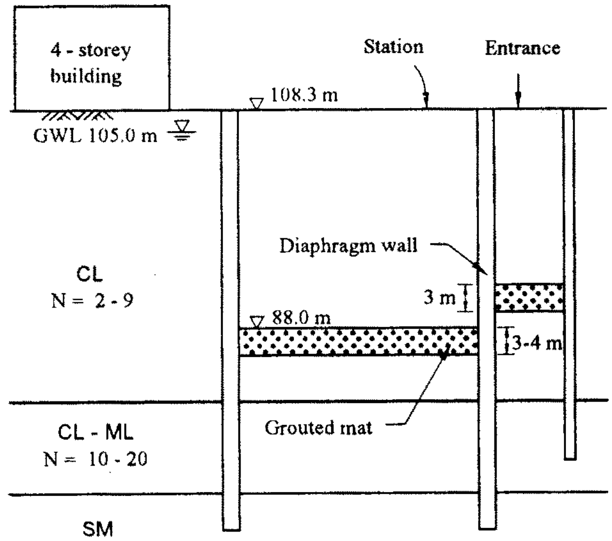

Fig. 2 Typical section of ground treatment

Four different jet grouting methods were applied for forming the grouted mats. With the patent names of JG (Jet Grout), JSG (Jumbo Special Grout), UHPG (Ultra High Pressure Grout) and SWING (Spreadable Wing), these methods can be categorized into the single-tube and the double-tube techniques. The basic principles and typical arrangement of the drilling and grouting procedures have been reported by researchers such as Miki (1985), Kauchinger and Welsh (1989).

The SWING method is a combination of mechanical mixing by wing bit and jet grouting. This method was described by Kawasaki et al. (1996). The blade for mechanical mixing is 2.0 m in height and 0.55 m in width. After sinking to the bottom of a hole of 600 mm in diameter, the blade turns 90° to a horizontal position. Nozzles for jet grouting are equipped at both sides of the blade. With the design diameter of 2.5 m, the inner zone of 2.0 m in diameter is formed by mixing and the outer rim of 0.25 m in thickness is formed by jet grouting.

The jet grouting parameters for Stations BL 13 to BL 15 are summarized in Table 2. Jet grout piles with diameters ranging from 2.55 m to 1.2 m were used. The grouted mats were mainly formed by piles with the larger diameters. The 1.2 m diameter piles were provided along the perimeters to fill the gap between the diaphragm walls and the larger diameter piles.

4.0 TESTING METHODS

The quality of ground treatment was evaluated by the core drilling, the unconfined compression (UC) tests conducted on the cored specimens, the cone penetration tests (CPT) and the pressuremeter tests (PMT).

(1) Core Drilling and Unconfined Compression Tests

The N-size drillings were conducted at the overlaps of jet grout piles in the period of 12 to 22 days after the completion of grouting. The cores were logged and

Table 2: Summary of jet grouting parameters

Grouting Method	Grouting Technique	Design Diameter	Spacing of Grout Piles	Injecting Pressure MPa		Rate of Injection	Withdrawal Rate	Rotation Speed	Grout Mix in Weight
		m	m	Cement Grout	Air	ℓ/min	min/m	rpm	C/W
JG	Single tube	1.20	1.04	40	-	200	4.2	5-8	0.83
JSG	Double tube	2.55	2.20	40	0.7	200	12.5	2-3	0.90-0.83
UHPG	Double tube	1.60	1.20	30-45	0.8	60	6.7	10-12	1.0
SWING	Mechanical mixing & double tube	2.50	2.15	20	0.7	55-65	5.0-6.7	15	1.4

the core recoveries were recorded. The actual core recoveries ranged from 91 % to 97 %, with an average of 94 %. The specimens were sent to the laboratory for determing their unconfined compressive strengths.

(2) Pressuremeter Tests

The pressuremeter test is an in-situ test which can measure the stress-strain relationship of the surrounding soil mass. Totally 10 pressuremeter tests were conducted at the 3 sites.

If the grouted mat can be idealized as a linear elastic/perfectly plastic body, the shear modulus and the undrained shear strength of the grouted soil mass can be estimated from the relationship between the pressure in the bulb and the volume change of the bulb. The testing apparatus used was a Menard Pressuremeter Type G-Am, a pre-boring type manufactured by the ROCTEST Company.

(3) Cone Penetration Tests

Totally forty cone penetration tests were conducted in the 3 grouted mats. The cone was 10 cm^2 in sectional area. The truck-mounted apparatus was manufactured by Hogentogler. Initially the cone penetration tests were conducted in the period of 3 to 10 days after completion of grouting. However, by that time the cone could no longer penetrate into the grouted soil mass. The rods started to buckle at the cone resistance varied from 13 MPa to 20 MPa. Only the surface of the grouted mat could be tested. In order to probe the full depth of the grouted mats, the tests were conducted on, as early as, the first or the second day after jet grouting.

5.0 INTERPRETATION OF TEST RESULTS

The undrained shear strengths of the jet grout piles estimated from the unconfined compression tests, the cone penetration tests and the pressuremeter tests, are denoted by $s_{u(UC)}$, $s_{u(CPT)}$, and $s_{u(PMT)}$, respectively.

5.1 UNCONFINED COMPRESSION TESTS

The relationship between the unconfined compressive strength, q_u, and the undrained shear strength, $s_{u(UC)}$, is represented by the equation:

$$s_{u(UC)} = \frac{q_u}{2} \qquad (1)$$

Sixty intact cored specimens were tested in the period between the 13th and the 22nd days after installation of the jet grout piles. However, the exact time of testing were only recorded for 18 specimens.

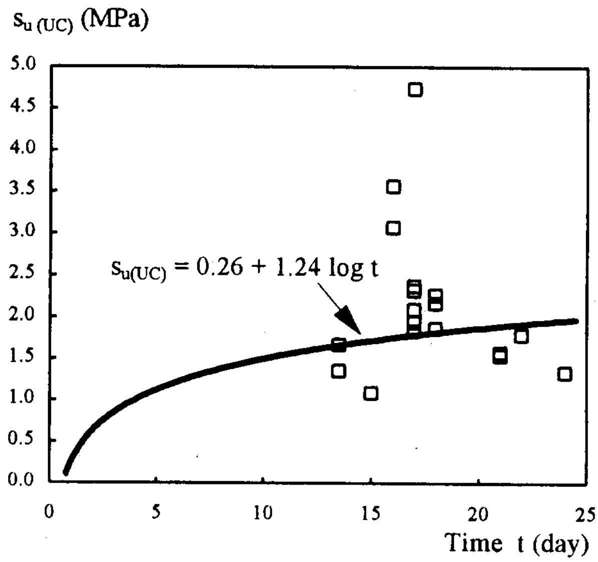

Fig. 3 : Results of UC tests

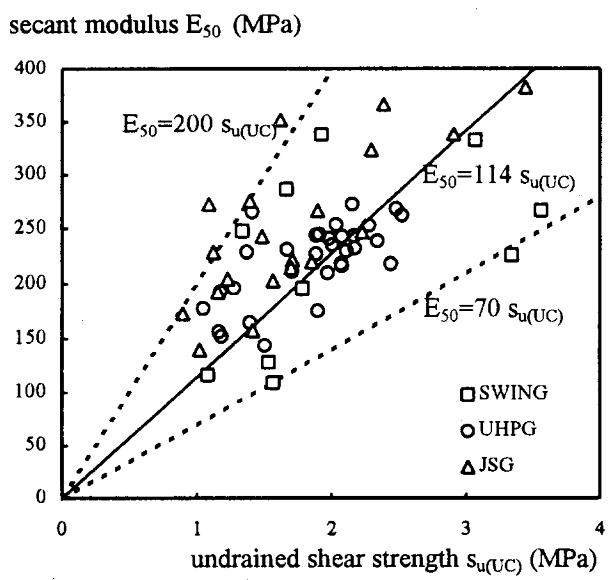

Fig. 4 : Relationship between $s_{u(UC)}$ and E_{50}

The $s_{u(UC)}$ values which are obtained from the unconfined compression tests are presented in Fig. 3, showing a trend of gaining strength with time. Ignoring the 3 extreme values which the $s_{u(UC)}$ values are greater than 3 MPa, the correlation of the average value of $s_{u(UC)}$ with time can be expressed by the regression curve as follows:

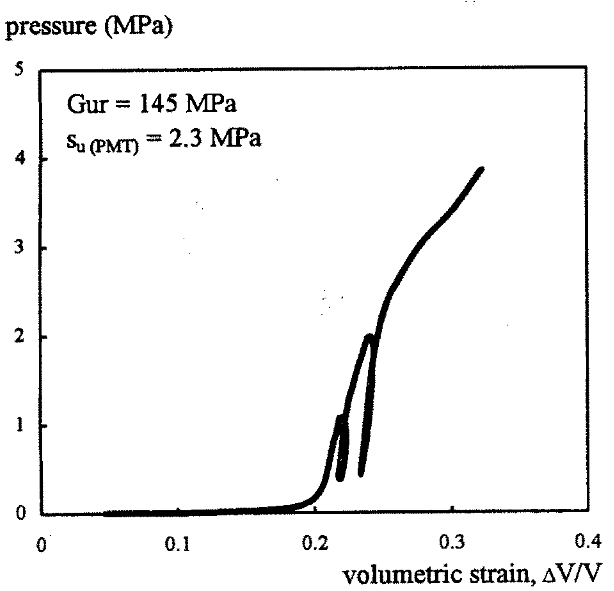

Fig. 5 : Typical pressure-volumetric strain plot of the PMT tests

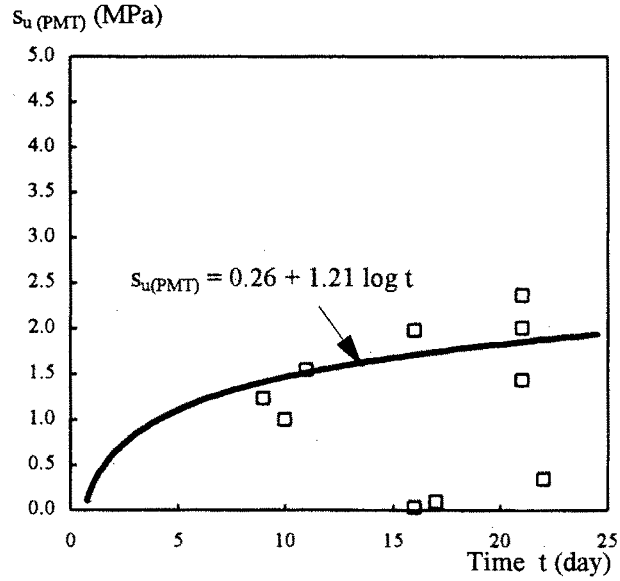

Fig. 6 : Results of PMT tests

$$s_{u(UC)} = 0.26 + 1.24 \ \log t, \text{ MPa} \quad (2)$$

where:

t = time in day after grouting and $1 \leq t \leq 24$.

The $s_{u(UC)}$ value of 0.26 MPa at $t=1$ day is obtained from the CPT tests as described in Section 5.3.

The peak strengths of the UC tests occurred at strains varying from 1 % to 3 %. The secant moduli of the 60 specimens, E_{50}, defined at the strain corresponding to the 50 % of the peak strength, are presented in Fig. 4. The $E_{50}/s_{u(UC)}$ values vary from 70 to 200 and the average value of E_{50} can be expressed as:

$$E_{50} = 114 s_{u(UC)} \quad (3)$$

Figure 4 shows that there is no significant difference in the shear strength or in the stiffness of the grouted mats formed by the 3 different ground treatment techniques.

5.2 PRESSUREMETER TESTS

According to Gibson and Anderson (1961), and assuming the grouted soil mass behaved ideally as an elastic-perfectly plastic medium, the undrained shear strength, $s_{u(PMT)}$, can be estimated from the limit pressure, P_ℓ, which was extrapolated from the pressure versus volumetric strain curve of the pressuremeter test as follows:

$$s_{u(PMT)} = \frac{p_\ell - \sigma_{ho}}{1 + \log e \dfrac{G_{ur}}{s_{u(PMT)}}} \quad (4)$$

and

$$G_{ur} = V \frac{dP}{dV} \quad (5)$$

where:

σ_{ho} = the in-situ total horizontal stress
G_{ur} = the shear modulus interpreted from the unload-reload cycle
V = the volume of the cylindrical cavity
P = the measured pressure

Figure 5 shows a typical pressure-volumetric strain plot of a PMT test conducted in the grouted soil. The results of the $s_{u(PMT)}$ values are shown in Fig. 6, showing the variation of strength with time. Ignoring the 3 values which are less than 0.3 MPa, a regression analysis on the 7 PMT tests gives the equation as follows:

$$s_{u(PMT)} = 0.26 + 1.21 \ \log t, \text{ MPa} \quad (6)$$

The G_{ur} values, interpreted from the unload-reload cycles conducted at volumetric strains ranging from 22% to 34%, are plotted against the $s_{u(PMT)}$ values in Fig. 7. Ignoring the 3 sets of data having the G_{ur}

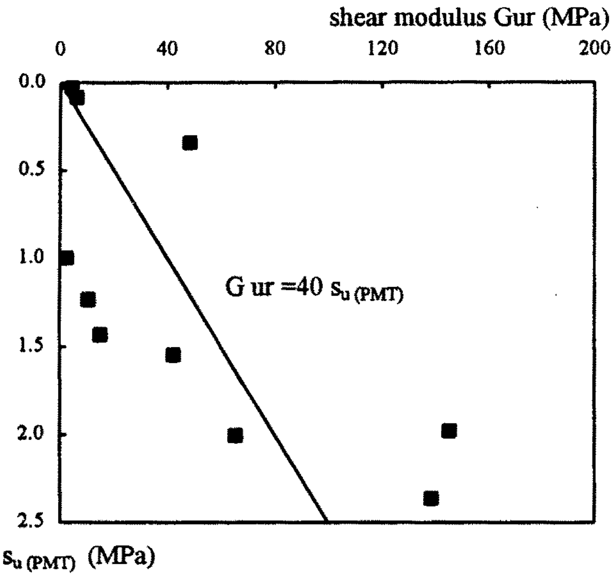

Fig. 7 : Relationship between $s_{u(PMT)}$ and shear modulus

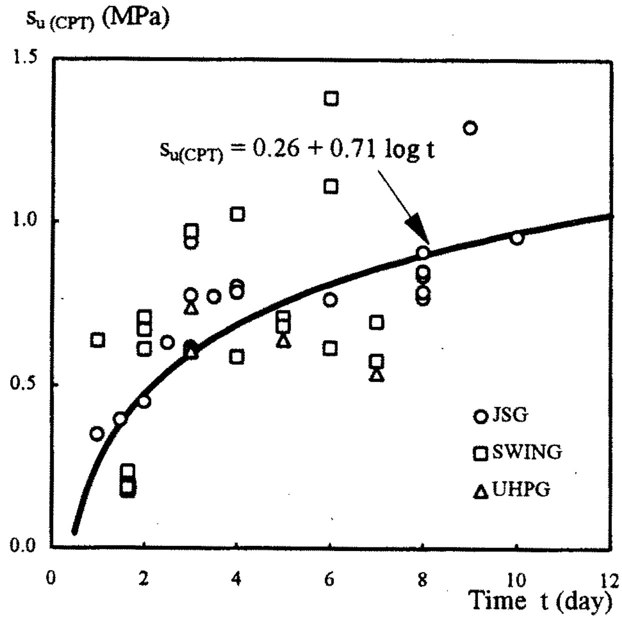

Fig. 8 : Results of CPT tests

values less than 2.6 MPa or the $s_{u(PMT)}$ values less than 0.1 MPa, the ratios of G_{ur} to $s_{u(PMT)}$ range from 10 to 70. The average G_{ur} value can be correlated with the $s_{u(PMT)}$ value as follows:

$$G_{ur} = 40 s_{u(PMT)} \quad (7)$$

The elastic moduli, E_{ur}, of the grouted mats can be calculated from the equation:

$$E_{ur} = 2(1+\nu) G_{ur} \quad (8)$$

where ν is the Poisson's ratio. Assuming $\nu = 0.35$ for the grouted soil mass and according to Eqs. 7 and 8:

$$E_{ur} = 108 s_{u(PMT)} \quad (9)$$

Comparison of Eqs. 3 and 9 suggests that the secant moduli, E_{50}, interpreted from the unconfined compression tests at 1 % to 3 % strains, are equivalent to the elastic moduli, E_{ur}, interpreted from the unload-reload cycles of the pressuremeter tests.

5.3 CONE PENETRATION TESTS

It is a common practice to determine the shear strengths of cohesive deposits from CPT by the empirical equation:

$$s_{u(CPT)} = \frac{q_c - \sigma_{vo}}{N_c} \quad (10)$$

where:

q_c = the cone resistance
σ_{vo} = the total overburden pressure
N_c = the cone factor which is usually calibrated by means of field vane or unconsolidated undrained triaxial tests.

Sufficient CPT data are available for the estimation of the N_c values, such as those reported by Lunne and Kleven (1981) and Tanaka (1995). In clays, the N_c values typically range from 8 to 20, with an average of about 15. Based on two sets of CPT and PMT tests which were both conducted on the 9th and the 10th days after jet grouting and assuming the $s_{u(CPT)}$ was equal to $s_{u(PMT)}$, the back-calculated N_c value for the grouted soil was 12.

Adopting a N_c value of 12, the estimated $s_{u(CPT)}$ results are presented in Fig. 8. There was no significant difference in the strength of the material improved by various methods of grouting. Moreover, based on 10 CPT tests conducted in 40 hrs after grouting at the depths of 16.4 m to 18.8 m, the 1-day $s_{u(CPT)}$ values ranged from 0.21 to 0.28 MPa, with an average of 0.26 MPa, indicating the uniformity along the depth of the treated soil.

For the CPT tests conducted in the period between 2 and 10 days after grouting, the cone did not penetrate through and was stopped at the top of the grouted mat. The q_c values ranged from 9.5 MPa to 20 MPa and the corresponding $s_{u(CPT)}$ values ranged from 0.8 MPa to 1.4

MPa. With a trend of gaining strength with time, a regression analysis of the 48 CPT tests gives the empirical equation as follows:

$$s_{u(CPT)} = 0.26 + 0.71 \log t, \text{ MPa} \quad (11)$$

Since the majority of the CPT tests were carried out at the surface of the grouted mat, the $s_{u(CPT)}$ results in Fig. 8 were basically the lower bound values of the grouted soil.

5.4 DISCUSSIONS

Comparison of Eqs. 2 and 6 indicates that the $s_{u(PMT)}$ values are approximately equal to the $s_{u(UC)}$, despite some extreme values were ignored during the statistical analyses. The slightly difference in the measured shear strengths is associated with the fact that the UC tests were conducted on the better part of the cores and were not representative of the grouted mass which could be non-homogeneous. On the contrary, the PMT tests were conducted not only in the better part but also in the weaker zone of the treated ground. Therefore, the in-situ PMT tests are more representative than the laboratory UC tests.

Comparison of Eqs. 2, 6 and 11 shows that the $s_{u(CPT)}$ values are only 60% of the $s_{u(PMT)}$ or the $s_{u(UC)}$ values. The difference is related to the limitation of the capacity of the cone penetrometer. Apparently the undrained shear strengths of the grouted mats are too high to be tested thoroughly by the CPT apparatus. Nevertheless, the results represent the lower bound values of the undrained shear strength of the grouted mass.

6.0 CONCLUSIONS

Based on the case histories of testings in 3 grouted mats, the following is concluded:

(1)There were no significant differences on the soil strength and the soil stiffness improved by different methods of jet grouting which include the single-tube, the double-tube and the combined mechanical mixing and jet grouting techniques.

(2)Based on 2 sets of CPT and PMT tests, an empirical cone factor of 12 for correlating the undrained shear strength with the cone resistance was back-calculated.

(3)Based on the unconfined compression tests conducted at strains varying from 1 % to 3 %, the average secant modulus to the undrained shear strength ratio for the jet grouted soil is 114.

(4)The average shear modulus to the $s_{u(PMT)}$ ratio for the grouted soil is 40. The G_{ur} values were interpreted from the unload-reload cycles conducted at the volumetric strains of 22 % to 34 %.

(5)The jet grouted soil had a trend of gaining strength with time. The average 1-day to 21-day undrained shear strengths can be represented by the empirical Equation 6.

7.0 ACKNOWLEDGMENTS

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