Construction Effect on Load Transfer along Bored Piles

Ming-Fang Chang, P.E., M.ASCE,[1] and Hong Zhu[2]

Abstract: The load transfer behavior along bored piles is affected by details of pile construction particularly those imposing stress and moisture changes to the surrounding soils. An investigation involving moisture migration tests, in situ horizontal stress measurements, and borehole shear and pressuremeter tests shows clear effects of construction that lead to subsequent changes in soil properties. The construction of bored piles in Singapore and the region often involves casting of concrete either in unsupported ''dry'' boreholes or in ''wet'' boreholes filled with water. It is necessary to differentiate these two extreme construction conditions in bored pile design. Based on triaxial compression and pressuremeter tests on the residual soil of the Jurong Formation in Singapore, the variation of soil modulus with shear strain can be described by a hyperbolic function. A procedure is recommended for assessing the combined effect of stress relief and soaking on soil modulus by introducing a modulus reduction factor. Modulus degradation curves from pressuremeter tests with the borehole conditions properly simulated are found capable of producing load transfer curves that are comparable to those deduced in the field.

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Introduction

Large diameter bored piles, also known as drilled shafts, are commonly used for the support of heavy loads in stiff to hard clay worldwide and in intermediate geomaterials such as residual soils and weathered rocks in tropical regions. Traditional methods of design based on static formulas and soil parameters derived from laboratory tests, or unit shaft and base resistance estimated from results of in situ tests, such as the penetration resistance, *N*, from the standard penetration test, have not been very successful. An improved design method is possible by making use of the load–settlement relationship predicted from site-specific load transfer curves using the load–transfer analysis (e.g., Reese 1978; Reese and O'Neill 1988; Chang and Goh 1989; Chang and Broms 1991). This method of design relies on careful evaluation of load–transfer curves from site investigation data and a proper account of construction details.

In Singapore, Malaysia, and other countries in the region, the construction of bored piles in residual soils and other intermediate geomaterials (O'Neill and Reese 1999) typically involves excavation of a borehole and casting of concrete in the hole. Because intermediate geomaterials are generally stiff to hard and cohesive, a rotary rig equipped with a short-flight auger is usually employed in excavation without the introduction of drilling fluid. In most cases where the ground is stable and the water table is low, boreholes may remain unsupported and stay ''dry.'' In other cases involving high water table and pervious strata, the borehole may become ''wet'' because of inflow of water and stabilization of boreholes by means of steel casing or drilling fluids such as bentonite slurry is necessary. Very seldom is drilling fluid introduced during excavation to assist the drilling process from the beginning.

Casting of concrete is carried out by direct pouring when the borehole is ''dry'' or by the tremie method when the borehole is ''wet.'' Usually, high slump concrete mixes are used, and/or suitable additives added, to avoid problems that may affect pile integrity.

One unique practice among the piling contractors in Singapore and Malaysia is to pour water into the borehole after inflow of water is identified, usually close to the bottom of the borehole or after the completion of excavation, and the borehole becomes ''wet.'' In this case, the borehole could be soaked in water before the arrival of ready-mix concrete for a varied period of time. The effect of ''wet'' construction may not always be critical, particularly if proper drilling slurry is used. Experience from borehole excavation using mineral slurry (or polymer fluids in recent years) in the United States has indicated that, in clayey soils, the load transfer developed along the shafts are comparable to those developed in shafts constructed in the ''dry'' (Reese and Touma 1972). Nevertheless, soaking of borehole under water after completion or near completion of excavation that is pertaining to the local practice represents an extreme condition that will lead to a lower bound to the load transfer expected in various ''wet'' construction methods and deserves a review and careful investigation.

Yong (1979), by studying model piles (38 mm in diameter) formed in kaolin, reported a 10% reduction of unit shaft resistance for a 12 h delay and 20% for a 27 h delay, compared with that with insignificant delay before concreting.

Balakrishnan et al. (1999) reported results of a study on a large number of instrumented bored piles, some constructed by ''dry'' excavation and others by ''wet'' excavation. These piles were

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installed in the residual soils and weathered rocks of the sedimentary Kenney Hill Formation of Malaysia, which is geologically related to the Jurong Formation of Singapore. The ground consists predominantly of sandy or clayey silt with N ranging from 11 to over 100 blows/0.3 m. Two distinctive groups of $t-z$ data were found among these bored piles excavated using short flight augers and concreted in ''dry'' and ''wet'' boreholes. Balakrishnan et al. (1999) found that the borehole condition appeared to have a significant effect on the shape of the load transfer curve, but not the limit unit shaft resistance f_{su}. A closer examination of the data for two specific sites where both ''dry'' and ''wet'' excavation boreholes were present, however, reveals a rather consistent reduction of the f_{su}/N ratio of typically 10–60% from wetting.

Yu (2000) load tested eight numbers of miniature bored piles on the Nanyang Technological University (NTU) in Singapore. The site located on the top of hill cut is underlain by stiff, brownish silty sand [liquid limit (LL)=32; plastic index (PI)=6] with a fines content (-75 μm) of 40% and a natural moisture content w_0 of 19.0% in the upper 0.7–1.0 m and yellowish and brownish silty clay (LL=34; PI=15) with a fines content of 60% and w_0 of 17.5% immediately blow the top layer. These piles, with a diameter of 0.10 m and lengths ranging from 1.0 to 2.0 m, were installed by using a hand-operated power auger. A number of piles were cast by direct pouring of cement mortar (water: sand=1:3) with a water/cement (w/c) ratio of 0.5 after completion of drilling in ''dry'' holes, and others in ''wet'' holes soaked under water for a specific period of time. From pullout tests, Yu (2000) found that f_{su} decreased with the soaking time for pile sections embedded in both soil layers. The f_{su} value decreased by 5–10% after soaking for 0.5–2 h, and by as much as 20% after 24 h.

To study the influence of different construction practices on pile behavior, a systematic investigation of effect of construction details, in particular those related to stress changes associated with delay in construction and wetting from soaking of boreholes in water, on the load transfer along bored piles is important. Investigations by means of in situ soil tests, laboratory moisture migration and shear tests, and monitoring of stress changes during a bored pile construction will be useful. Of great importance will be a close simulation of two typical extreme construction conditions involving ''dry'' and ''wet'' boreholes. Results of these investigations should help to verify if the influence of construction on load transfer that have been previously observed is directly relevant for bored piles in intermediate geomaterials or in weathered rock Formations in the tropics.

The focus of this investigation is on construction effect, particularly the combination of stress and moisture changes from delayed construction and wetting from direct contact of borehole with water, on the load transfer characteristics along bored piles.

Stress and Moisture Changes in Surrounding Soils

The load transfer along a pile depends directly on the prevailing horizontal effective stress, the soil shear characteristics and the roughness of the pile-soil interface. The interface roughness, which depends on the construction machinery and techniques, usually does not vary significantly from one pile to another on a given site (O'Neill 2001). The effective horizontal stress and the soil shear characteristics are controlled by the in situ stress condition in the ground and the subsequent changes from pile construction details, such as time taken for boring and duration of borehole opening, and the presence of water in the borehole prior to concreting. If the borehole is soaked in water, moisture increase in the surrounding soil will lead to deterioration in soil stiffness and soil strength. Further migration of moisture and stress changes will occur during and right after casting from the action of wet concrete and subsequent pozzalanic effect cum hydration of cement during curing. These physical changes associated with the various phases of construction will affect the detailed load transfer mechanism and the behavior of bored piles and deserve a brief review.

Changes from Boring

From borehole excavation, a relatively narrow zone of soil surrounding the borehole will undergo remolding. The size of the remolded zone varies with the drilling technique and the soil properties. Lord (1989) found a remolded zone about 20–30 mm thick along bored piles in slightly to highly weathered chalk.

Marsland and Randolph (1977) observed a drop of E_s/s_u value (E_s=equivalent linear elastic modulus; s_u=undrained shear strength) from 500 a short time after excavation to 100–200 after a delay of over 8 h at the bottom of pile shafts in London Clay. O'Neill (2001) reported that the effect of the stress relief could extend to 2–3 borehole radii away from a drilled shaft in Beaumont Clay, and the shear wave velocities in the direction perpendicular to the pile shaft at the concrete–soil interface were only about 70% of those in the free field approximately 3 h after the borehole was open.

Changes from Casting

Lings et al. (1994) reported that when the borehole was filled with concrete with a high w/c ratio, the concrete pressure would follow the hydrostatic line at shallow depths, but increase at a reduced rate at greater depths in diaphragm wall construction. The pressure distribution against a circular borehole in pile construction, however, could be different due to differences in geometry and arching effects. Indeed, O'Neill and Hassan (1994) reported that the total lateral concrete pressure exerting on the borehole face at the time of concrete placement, in the case of drilled shafts, was probably nearly equal to the normal radial interface stress at the time of loading.

Yong (1979) observed a change in moisture content in kaolin of up to 60 mm from the pile–soil interface, with the maximum increase measured a few days after concreting. Chuang and Reese (1969), based on tests on remolded Beaumont clay, found that w_0 in the soil was important, but the w/c ratio was a key factor in water migration. Clayton and Milititsky's (1983) study in remolded London clay showed that the moisture content in the vicinity of concrete depended largely on the w/c ratio.

Beech and Kulhawy (1987) observed the dependency of moisture migration on w_0 relative to the LL and the plastic limit (PL), as well as the plasticity index (PI) of the soil. The magnitude of the water content change increased as PI increased and when w_0 is close to PL.

In the field, Meyerhof and Murdock (1953) observed that, in London clay (liquidity index LI≈0.04), the water content was 2–7% higher than w_0 at the shaft face of a bored pile and gradually decreased over a distance of 51 mm. However, O'Neill and Reese (1970) observed no definite trend in the vicinity of two shafts in Beaumont clay.

Changes from Curing

The concrete pressure exerted on the borehole wall will decrease as the concrete starts to set, resulting in the redistribution of stresses around the bored piles. The magnitude of the horizontal stress at the final stage will depend on the interaction between concrete and the soil in contact. In addition, temperature change due to heat of hydration and pozzalanic action may also have an effect.

Field measurements on a 4.5 m long diaphragm wall constructed in a stiff soil by Uff (1969) showed a pressure reduction of about 30% 8 weeks after concreting. Uriel and Oteo (1977) observed in the construction of a 3.4 m long wall panel an average reduction of 20% 24 h after casting. Lings et al. (1994) reported an average reduction of 25% on an 8.5 m long diaphragm wall in stiff to very stiff Gault clay 10 days after concreting.

It should be noted that the stress at the pile–soil interface can decrease or increase depending on the details of concrete mix, the suction potential in the surrounding soil, the temperature of the ambient soil, and the rate of the placement of concrete. The observed pressure often varies and an independent verification by direct investigation adjacent to a pile shaft will be useful.

Laboratory Investigation of Moisture Migration

A series of tests were conducted in the laboratory to study the effect of w_i in the soil and the w/c ratio of the cement mortar on the magnitude and pattern of moisture migration in the residual soil of the Jurong Formation. The soil samples were taken from a bored pile construction site on NTU campus in Singapore. The soil (LL=36.3, PL=19.5; $w_0=18.9\%$), with a fines content of 61.4%, is classified as silty clay.

The soil was first air dried and sieved through U.S. No. 4 (2 mm opening) to remove the gravel size particles. A 100 mm high sample was prepared by manually compacting the soil layer by layer using a hand-held Standard Proctor hammer into a 200 mm long plastic pipe with an internal diameter of 95 mm. The sample were compacted until the specified height was reached and the resulting bulk density was 2.1 Mg/m^3. Mortar with a designated w/c ratio was then poured on the top of the sample. The two ends of the pipe were immediately wrapped with cling film and aluminum foil and dipped in wax. The specimen was weighed and stored in a humidity controlled room. After 7 days, sampling was then carried out using a 38 mm diameter "mini" sampler. The sample was subsequently sliced and the water content of each slice evaluated.

Samples with three different combinations of w/c ratios of 0.5, 0.6, and 0.7 and w_i values of 14.1, 17.0, and 22.2% were prepared in the tests. The corresponding degrees of saturation of the compacted soil samples were 82.8, 92.4, and near 100%, respectively. The suction pressures based on the average soil water characteristic curve for the residual soil under investigation (Leong et al. 2002) were estimated at 60 kPa, 10 kPa, and close to 0, respectively.

Fig. 1 shows the distribution of water content, w, in soil samples in contact with mortar of different w/c ratios. A variation of w with the distance from the soil–mortar interface is clearly observed for all tests with different w_i and w/c ratios. Except for the combination of w/c of 0.5 and w_i of 22.2% for which there was a consistent decrease in w throughout the soil sample, an increase in w is generally observed. The higher the w/c ratio, the more abundant is the water supply and the greater is the amount of increase in w. Similarly, the lower the w_i, the greater is the suction and the greater is the increase in w.

The maximum observed increase in w is typically 3% at the soil/mortar interface, similar to that reported by Clayton and Militsky (1983) for a mortar with $w/c=0.3$. The zone of influence

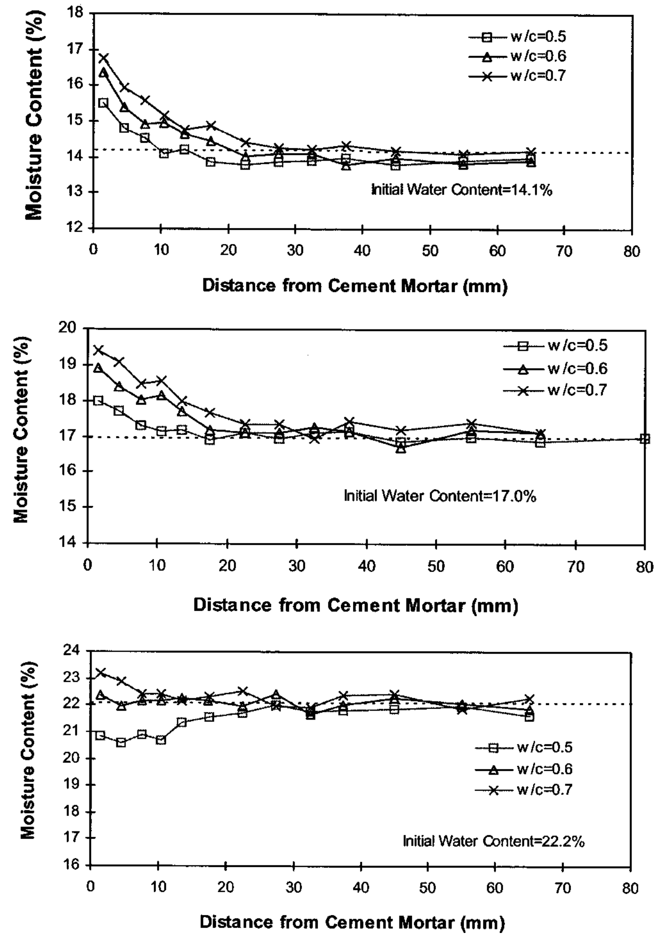

Fig. 1. Influence of initial moisture content and water/cement ratio on moisture distribution of compacted residual soil samples

within which a significant change in w was observed is seen to extend to around 20–30 mm from the interface, about half of that observed by Yong (1979) in the more pervious Kaolin and by Meyerhof and Murdock (1953) in the often fissured London clay.

Fig. 2 shows the average change in water content within first 20 mm from mortar plotted as a function of LI of the soil. It shows clearly that the demand of soil for water is stronger as LI becomes smaller, and consequently more water is absorbed by the soil, similar to that observed by Beech and Kulhawy (1987). Note that the final moisture distribution in the soil depends not only on the suction or demand of the soil for water, but also on that of the

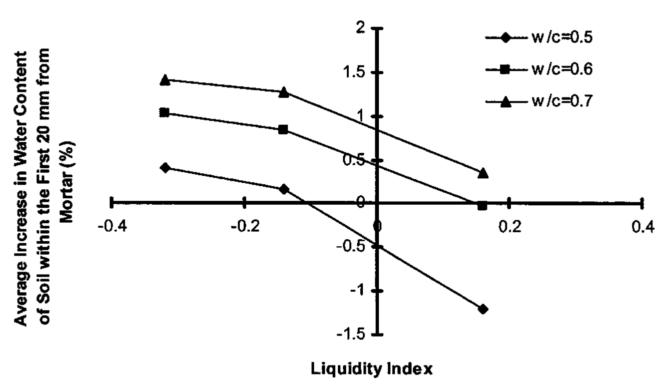

Fig. 2. Influence of liquidity index on moisture migration

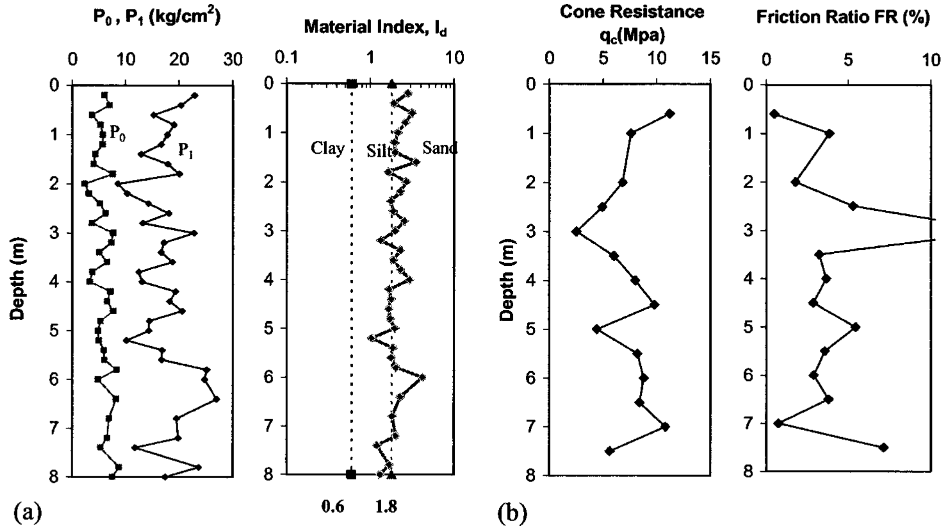

Fig. 3. Penetration response profiles in the compacted fill: (a) from dilatometer test and (b) from cone penetration test

mortar. The suction balance between soil and mortar may change with time, leading to further changes in w near the interface and consequently the interface shear characteristics.

Field Observation of Changes in Horizontal Stress

Field monitoring of changes in horizontal stress in the ground adjacent to a pile bore during the construction of a bored pile (diameter=0.8 m, length=30 m) was made at a piling site on the NTU campus. The upper part of the ground consisted of over 8 m of compacted residual soil fill with its water content similar to w_0 in the Jurong Formation that immediately underlies the compacted fill.

Prior to excavation, one DMT, using a standard dilatometer blade, and one CPT, using a mechanical cone, were carried out. The DMT results are presented as profiles of p_0 (membrane lift-off pressure) and p_1 (pressure that corresponds to a membrane displacement of 1.0 mm at the center), as shown in Fig. 3. The compacted soil can be classified as sandy silt according to Marchetti (1980) based on the material index, I_d, calculated from p_0 and p_1 pressures. The cone penetration resistance, q_c, from the CPT, also shown in Fig. 3, was found to fluctuate between 5 and 10 MPa. The friction ratio is typically between 3 and 5%, also indicating that the soil is rather silty.

Three flat dilatometers, arranged in a row on one side of the pile, were installed at 0.5, 0.9, and 1.7 m away from the borehole wall 5 days prior to pile construction and left in place. Their respective depths of embedment were 4.5 m (DMT-1), 6.0 m (DMT-2), and 8.0 m (DMT-3) within the compacted fill.

During excavation by augering, some localized collapses of the borehole wall were observed. In an attempt by the piling contractor to prevent further collapse of the borehole wall, the borehole was filled with water one hour after completion of excavation. Casting of concrete ($w/c=0.55$) was subsequently conducted by the tremie method after over 4 h of delay. Dilatometer p_0 pressures were taken at regular time intervals during the pile construction.

Fig. 4 shows the observed variations of p_0 pressure, normalized by the initial p_0 pressure, $(p_0)_i$, prior to excavation, with time from DMT-1 and DMT-2. For unknown reasons, DMT-3, registered a $(p_0)_i$ of 190 kPa, did not work after the initial reading was taken. The measured $(p_0)_i$ was 180 kPa from DMT-1, similar to that in DMT-3; but it was at a much higher value of 340 kPa from DMT-2, possibly due to the heterogeneous nature of the compacted fill. The measured $(p_0)_i$ corresponded to an earth pressure coefficient K of around 2 at DMT-1 level and 1.2 at DMT-3 level, withstanding the disturbance induced to the ground in the vicinity of the dilatometer blade.

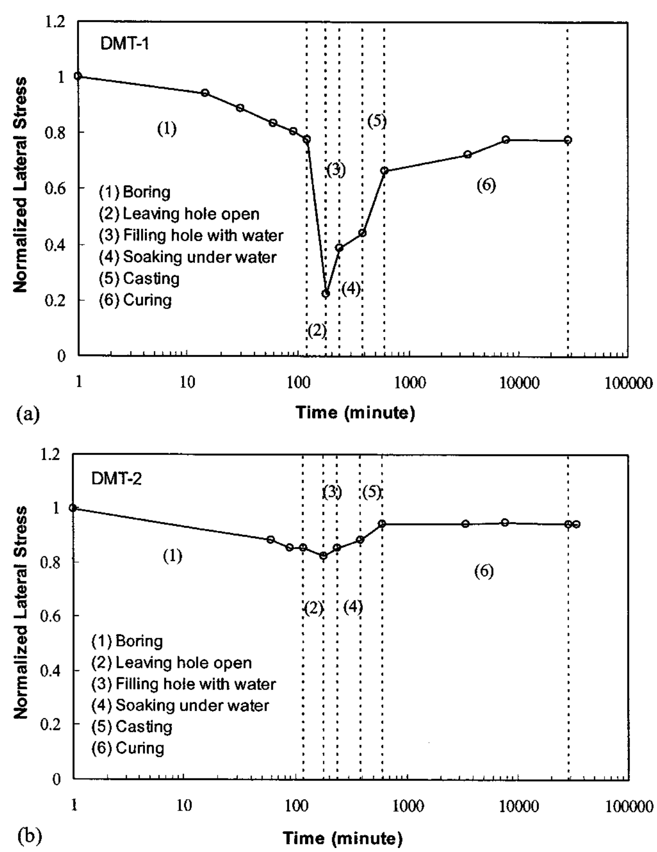

Fig. 4. Measured normalized lateral stress during pile construction from (a) dilatometer-1 and (b) dilatometer-2

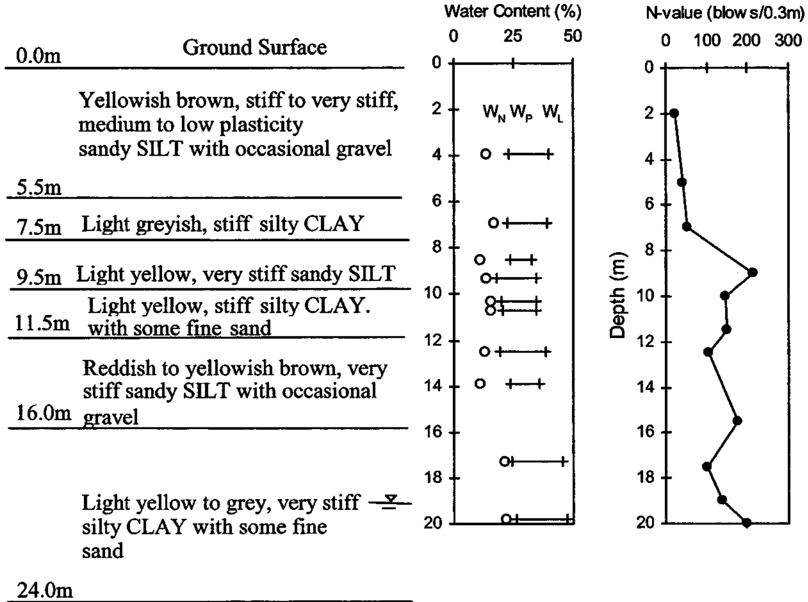

Fig. 5. Soil stratification and general soil properties at Nanyang Technological University test site

The change in p_0 pressure clearly shows that borehole excavation led to a significant reduction in horizontal stress in the soil around the borehole. The measured decreases in horizontal stress at the end of excavation were about 23 and 15% of $(p_0)_i$ at 0.5 and 0.9 m away from the excavation face, respectively. These reductions increased to 78 and 18% after the borehole was left open for 1 h, prior to pouring of water. Immediately after the borehole was filled with water, there was a partial recovery in p_0 and the reductions dropped to 60 and 15%, respectively. Interestingly, there was a further recovery in the p_0 pressure after the borehole was soaked for over 3 h possibly due to moisture changes in the soil adjacent to the membrane, and the reductions were maintained at 55 and 11%, respectively, just prior to casting.

The measured p_0 pressures from DMT-1 and DMT-2 at the completion of casting by tremie using 175 mm high slump concrete recovered to 67 and 94% of their respective original values. The horizontal stress in the compacted soil near the pile shaft did not return completely to the original values even 19 days after construction. The final equilibrium horizontal stress at 0.5 and 0.9 m from the excavation face were 80 and 94% of $(p_0)_i$, respectively. The common practice of assuming $K=K_0$ (the coefficient of earth pressure at rest) for bored and cast-in-place piles cannot be verified in this case. Neither is the similar extent of horizontal stress reduction after casting in diaphragm wall construction as reported by others observed here.

Field Investigation of Soaking Effect on Soil Characteristics

An investigation was carried out at the NTU test site to see the effect of soaking of boreholes on the properties of the residual soil in Jurong Formation. Fig. 5 shows soil stratification and the profiles of selected soil properties at the site. The groundwater table was found at around 19 m below the ground surface and the soil above the water table was generally unsaturated with w_0 $<$PL. Two types of in situ tests, the borehole shear test (BST) and the prebored pressuremeter test (PMT), were used in the investigation.

A borehole shear test apparatus (Handy 1981) was used for a rapid, qualitative assessment of the friction angle and the cohesion intercept of soils. In the BST, a set of shear plates with two sharp 30° half-wedge teeth at 25.4 mm apart, providing an equivalent shear surface of 645 mm^2, was adopted to ensure sufficient penetration in the stiff residual soils (Yu 2000). The test involved applying a normal stress against the borehole wall and measuring the maximum pulling force that is required to fully mobilize resistance along the shear head during pullout. The measurements were repeated at the same depth for typically a minimum of three normal stresses at three different orientations approximately 120° apart. Two series of BSTs, one in freshly prepared boreholes and the other in boreholes drilled and soaked for specified periods of time, were conducted. Both the time for consolidation of soil after normal stress application and that for the pull out were so selected that the test could be assumed drained. Typically, the consolidation time was about 5 min, and the pulling rate was 1 mm/min.

A single cell Japanese (OYO) type pressuremeter was used in the PMT. The probe, made of rubber membrane, was lowered down to the test depth in a borehole carefully prepared using the rotary wash boring technique. The membrane was then expanded by means of pressurized gas and the corresponding radial displacement was measured using two callipers linked to a displacement transducer. Through step-by-step applications of pressure, an expansion curve showing the relationship between the applied pressure and the radius of the borehole was obtained. Two comparative series of PMTs were conducted in two 70 mm in diameter boreholes. In the first series, the expansion test was conducted immediately after the borehole reached a predetermined depth, as it is done in practice, and was denoted as the "unsoaked" test. In the second series, the probe was kept in the borehole for a certain period of time after installation, with the borehole soaked in water before the expansion of membrane, and was denoted as the "soaked" test.

Table 1 summarizes the shear strength parameters, c' and ϕ', deduced from BSTs for the two distinctive soils, namely the sandy silt and the silty clay, present at the test site. Both soils are cohesive and with LI$<$0. With few exceptions, the decrease in c'

Table 1. Summary of Shear Strength Parameters from Borehole Shear Tests

Soil type	Depth (m)	Cohesion c' (kPa) Soaking time (h) 0	1	2	3	4	5	Friction angle ϕ' (deg) Soaking time (h) 0	1	2	3	4	5
Sandy silt	3.5	12.8	8.8	—	—	6.2	—	36.2	34.3	—	—	31.4	—
	8.0	14.9	—	5.6	—	—	—	29.8[a]	—	34.4	—	—	—
	12.0	10.6	5.4	2.0	—	—	—	24.0[a]	42.8	45.0	—	—	—
	13.5	13.9	—	4.6	—	2.8	—	36.7	—	35.0	—	38.6	—
Silty clay	6.0	15.3	—	8.6	—	—	—	28.8	—	—	26.7	—	—
	7.0	22.8	—	15.7	—	—	—	35.6	—	34.4	—	—	—
	10.0	26.4	—	9.6	—	6.8	—	42.0	—	36.0	—	35.9	—
	16.5	14.0	—	8.7	—	—	—	37.2	—	30.0	—	—	—
	19.0	21.0	—	—	—	—	12.4	39.6	—	—	—	—	31.0

[a]Exceptional values.

and ϕ' due to soaking is evident for both soils, particularly for c'. The reduction in c' after 1–4 h of soaking is 30–80% for the sandy silt and 30–70% for the silty clay. The reduction in ϕ' angle varies from negligible to around 20% for both groups of soils. Notwithstanding the fact that a translation of these changes into the change in shaft resistance is not that direct, the soaking effect is evident.

Fig. 6 shows typical expansion curves for unsoaked and soaked conditions from PMTs at a depth of 6.0 m. The pressuremeter shear modulus, G_p, was calculated from the slope of the initial linear portion based on cavity expansion theory (e.g., Mair and Wood 1987). The limit pressure p_L, the pressure required to double the cavity volume, was obtained from the expansion curve or by extrapolation. The net limit pressure, defined as $p_L^* = (p_L - \sigma_{v0})$, where σ_{v0} is the in situ total vertical stress, can be readily calculated.

Figs. 7(a and b) show, respectively, the G_p (typically at 1.5% cavity strain) and the p_L^* profiles from PMTs in both "unsoaked" and "soaked" boreholes. The soaking process has clearly affected G_p and p_L^* and the effect is more significant in the sandy silt than in the silty clay. Within the soaking duration of 2–5 h, the reduction in G_p ranges from 25 to 60% for the sandy silt and 17–50% for the silty clay. The reduction in p_L^* ranges from 20 to 60% for the sandy silt and 20–50% for the silty clay. Note that the observation based on the PMT represents an average response of adjacent soils from the combined effect of stress relief and moisture migration in the soil within a distance approximately equal the length of the probe, which is 0.6 m, as a result of soaking.

Effect of Stress Relief and Soaking on Modulus Degradation

The stress–strain behavior of natural soils is highly nonlinear and the elastic modulus generally decreases with the increase in shear strain. Extensive studies have shown that this degradation of soil shear modulus, G, with shear strain, ϵ_s, significantly influences the performance of a foundation system, especially in stiff soils (e.g., Jardine et al. 1986).

The modulus degradation of a residual soil can be expressed as a hyperbolic function as follows (e.g., Zhu and Chang 2002)

$$\frac{G}{G_{\max}} = 1 - f\left(\frac{G}{G_{\max}}\frac{\epsilon_s}{\epsilon_r}\right)^g \tag{1}$$

where f and g=fitting parameters and $\tau_{\max}$ and $G_{\max}$=maximum shear stress and maximum shear modulus, repectively; and ϵ_r=reference shear strain defined as $\epsilon_r = \tau_{\max}/G_{\max}$.

In the construction of bored piles, stress relief from excavation followed by soaking of boreholes will affect the shear modulus of the surrounding soils. Both the triaxial compression test with

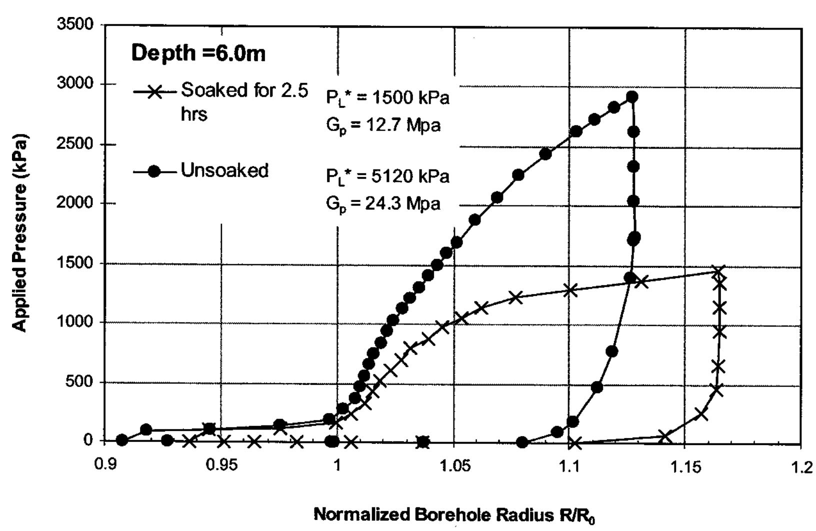

Fig. 6. Comparison of pressuremeter expansion curves for unsoaked and soaked conditions

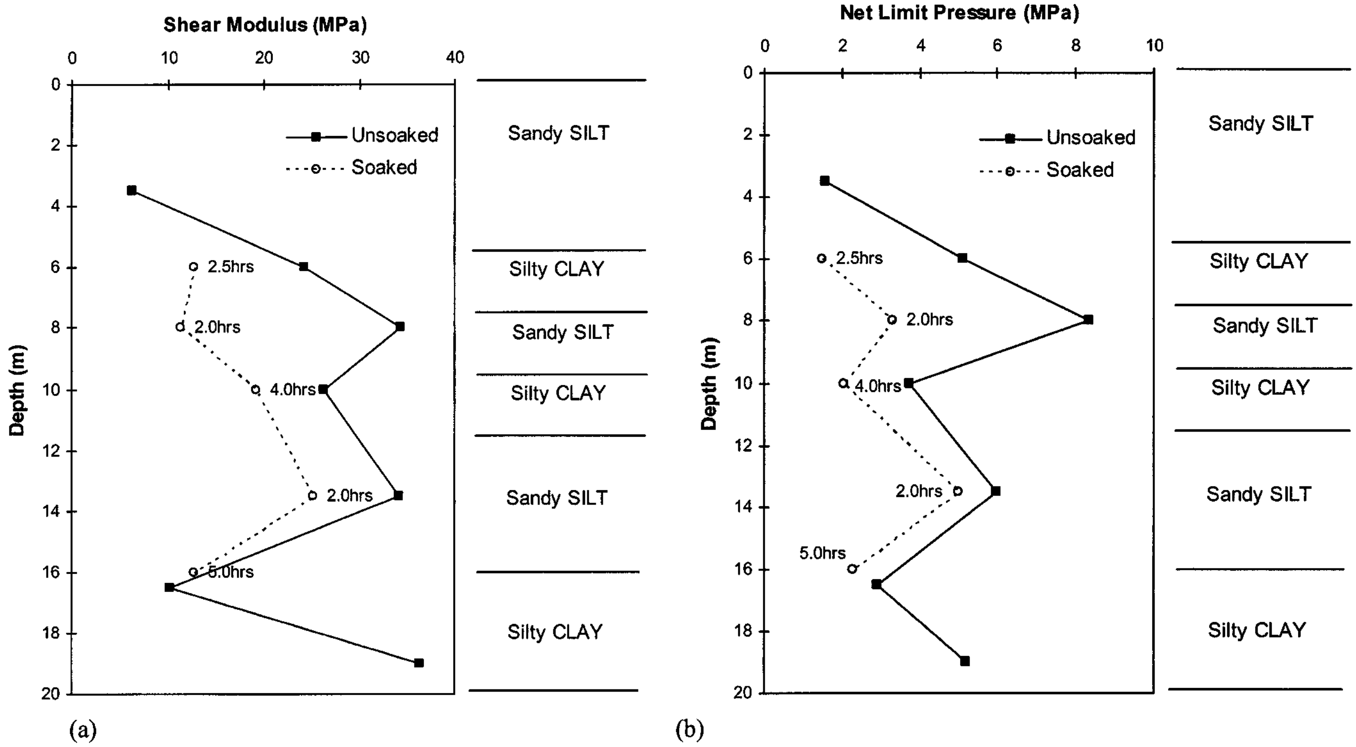

Fig. 7. Profiles from unsoaked and soaked pressuremeter tests: (a) Shear modulus and (b) net limit pressure

small strain measurements in the laboratory and the pressuremeter test that incorporated unloading–reloading in the field are useful for investigating such effects in the study.

A series of consolidated undrained (CU) triaxial compression tests were carried out on both "natural" and "presaturated" specimens of the silty clay and the sandy silt from the NTU test site (Fig. 5). "Undisturbed" samples obtained using a Mazier sampler were used. For the "presaturated" tests, the samples were first subjected to saturation under a selected back pressure and a confining pressure that produced an effective confinement equal to the initial suction in the sample. The initial suction measured using the null-type axis-translation technique (e.g., Fredlund and Rahardjo 1993) were typically 30–50 kPa for the samples. Both "natural" and "presaturated" samples were firstly isotropically consolidated to an effective pressure, σ'_c, equivalent to the mean in situ effective stress and subsequently unloaded to a mean effective stress that equals to $0.5\sigma'_c$ to simulate the stress relief due to a delayed borehole excavation. Subsequently, the sample was sheared under undrained condition and the mean effective stress was maintained at a constant value to simulate the expansion of a cavity as that in the pressuremeter test. Local displacement or strain measurements were carried out during shearing using submersible LVDTs.

It is interesting that the initial water contents, w_i, in the "natural" and the "presaturated" specimens prior to shear were 20.3 and 22.5%, respectively, for the silty clay and 12.9 and 16.2%, respectively, for the sand silt. This difference of 2–4% in w_i between the two types of specimens is similar to that observed near the soil–mortar interface in the moisture migration tests discussed earlier.

Fig. 8 shows the variation of the shear modulus, G, normalized by $G_{\max}$ with the shear strain, ϵ_s, for the two soils subjected to different saturation conditions. The reduction in soil modulus is significant, particularly when $\epsilon_s \leqslant 0.1\%$. Fig. 8 also shows how the modulus degradation data are fitted with hyperbolic curves as described by Eq. (1), with values of $G_{\max}$ obtained from the ultrasonic impulse velocity measurements, and the appropriate fitting parameters f and g. The parameter f appears unaffected by soaking and is at 0.98 for both types of soils. However, the parameter g for the "dry" condition is two times of that in the "wet" condition for the silty clay, and the corresponding ratio of g is 2.5 for the more pervious sandy silt.

The PMT is an ideal alternative for investigating the combined effect of stress relief and soaking of borehole in water on the soils adjacent to a bored pile, as the relevant borehole conditions can be simulated in a practical manner and the corresponding changes in shear modulus can be readily assessed. For this application, unload–reload cycles will need to be incorporated in the expansion test so that the reloading curve, which is believed to be representative of the in situ response of the soil, can be used to derive the modulus degradation curve.

Two series of comparative PMTs were carried out in both the silty clay and the sandy silt in two parallel boreholes, one soaked and the other unsoaked, at the NTU test site. Fig. 9 shows the PMT modulus degradation data corresponding to both unsoaked and soaked conditions in the silty clay at a depth of 10 m and in the sandy silt at a depth of 13.5 m, as derived by Zhu and Chang (2002). Soaking has clearly affected the modulus degradation curves for both soil groups, similar to that observed in the triaxial compression tests. Fig. 9 also shows the best fitting curves and the appropriate fitting parameters, with values of $G_{\max}$ estimated by assuming $G_{\max}/G_{\mathrm{ur}}=2.5$, where G_{ur} is the unload–reload modulus (Zhu 2000). From the best fits, the f parameter is found to be at a constant value of 0.98, the same as that observed in the triaxial CU tests. The g parameters from the PMTs, however, are consistently lower, at 40–60% of the triaxial values. It is believed differences in the simulated stress relief, mass of soil tested, and progressive failure are probably responsible for the observed difference between the degradation curves from the two types of tests. Nevertheless, the ratios of g between the "dry" and the

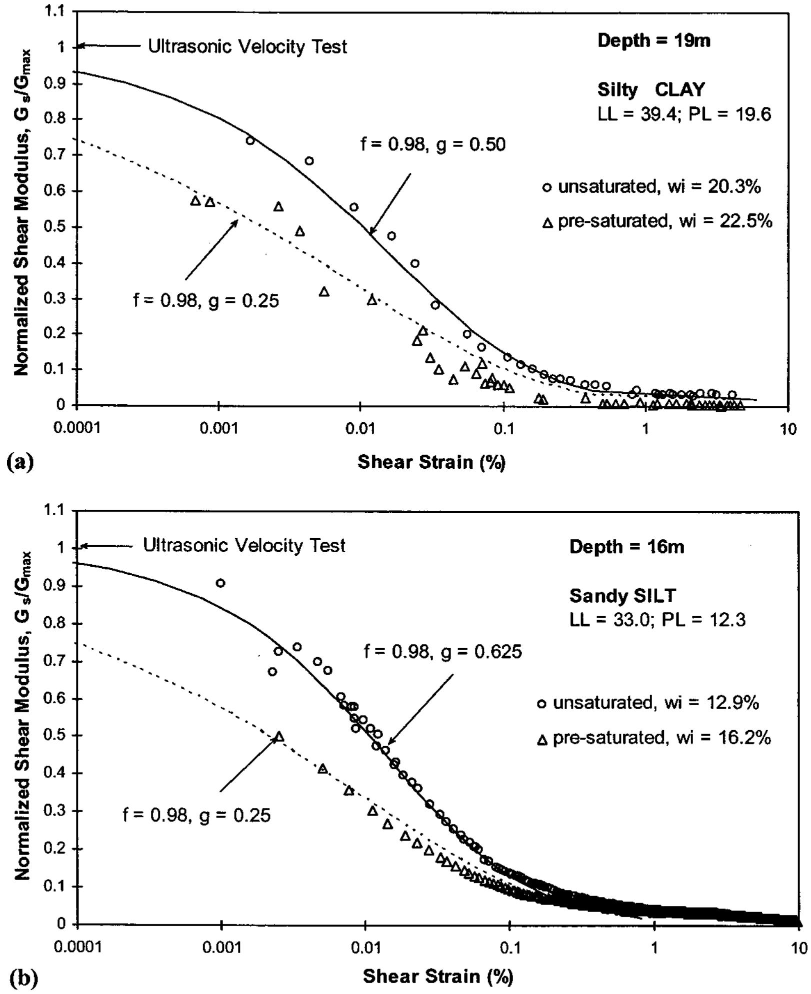

Fig. 8. Shear modulus degradation behavior from triaxial compression tests in: (a) Silty clay and (b) sandy silt

"wet" boreholes, which is 2.5 for the silty clay and 2.3 for the sandy silt, are similar to those from the triaxial tests. For evaluating the combined effect of stress relief and soaking of borehole on pile behavior, the PMT is preferred due to its closer simulation of the excavation condition in pile construction, even though fewer and often less consistent data points are expected when compared to the CU triaxial test.

It should be noted, by proper selection of different sets of fitting parameters of (f_2,g_2) and (f_1,g_1), respectively, Eq. (1) can match "natural" and "presaturated" test data from both the triaxial compression tests and the pressuremeter tests well, as illustrated in Figs. 8 and 9. It appears that the difference in saturation condition leads to a significant reduction in parameter g that controls the shape of the modulus degradation, but not in parameter f that signifies the magnitude of degradation in the Jurong Formation.

From Eq. (1), a factor reflecting modulus reduction can be deduced as follows:

$$f_G=1-\frac{1-f_1\left(\frac{\tau}{\tau_{max}}\right)^{g_1}}{1-f_2\left(\frac{\tau}{\tau_{max}}\right)^{g_2}} \tag{2}$$

where f_G=factor of modulus reduction; and g_1 and $g_2=g$ parameters corresponding to "presaturated" and "natural" borehole conditions, respectively. This modulus reduction factor can be used to evaluate the combined effect of stress relief and soaking on modulus degradation.

Fig. 10 shows the variation of f_G with stress level for different ratios of g_2/g_1. A single value of f_1 and f_2 of 0.98 that appeared to be a characteristic that is typical of the cohesive residual soil of the Jurong Formation, was assumed. The modulus reduction factor f_G is seen to increase as the g_2/g_1 ratio increases. The reduction factor f_G which is relatively independent of the stress ratio is typically between 0.2 and 0.7.

Note that from triaxial compression tests, the g_2/g_1 value is around 2.0 for the silty clay, implying that saturation would lead to a modulus reduction of about 40%. Similarly, the corresponding g_2/g_1 value of 2.5 for the sandy silt implies that the modulus reduction would be around 50%, which is slightly larger that observed in the sitly clay. Similar extents of reduction in shear modulus were also observed in the PMTs based on the observed g_2/g_1 ratios of between 2.3 and 2.5 for two different soils, as well as from the profile of G_p presented earlier in Fig. 7, although the various G values may correspond to different levels of shear strain.

Effect of Stress Relief and Soaking on Load Transfer Curves

Zhu and Chang (2002) proposed a procedure using modulus degradation curves as obtained from PMTs for evaluating load transfer $(t-z)$ curves along piles, on the basis of Randolph and

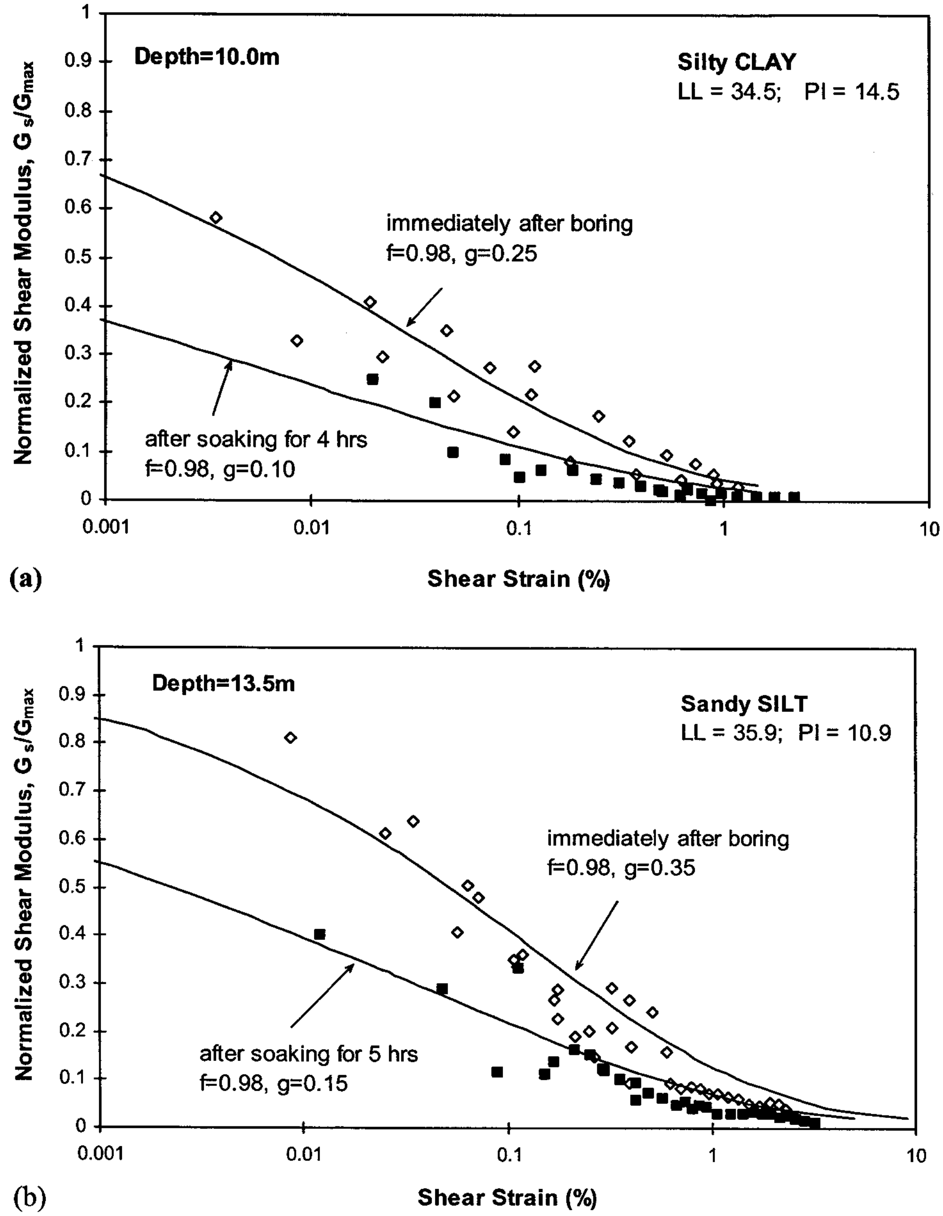

Fig. 9. Modulus degradation in different test conditions from pressuremeter tests in: (a) silty clay; (b) sandy silt

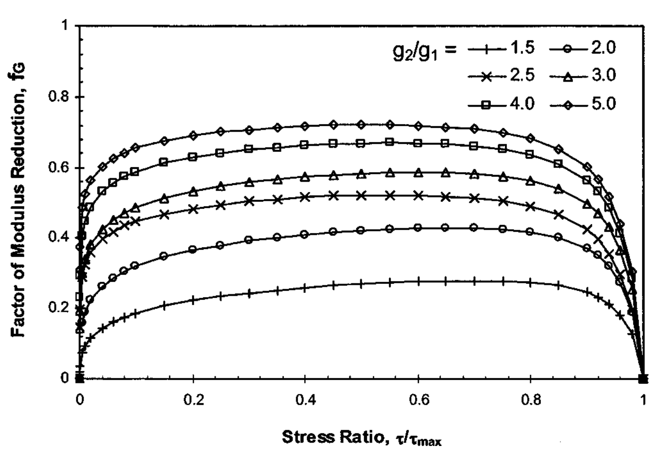

Fig. 10. Variation of modulus reduction factor with stress level

Wroth's (1978) framework for pile analysis. The procedure was adopted in this study for investigating the effect of stress relief and soaking on load transfer along bored piles. The key parameters required in the analysis are the pile slenderness ratio L/d (L=length; d=diameter), the mid-level to pile base modulus ratio or the inhomogeneity factor ρ, and the degradation curve fitting parameters f and g.

Field observed load transfer data from a large number of bored piles, instrumented with strain gauges at different levels, constructed in the Kenny Hill Formation in Malaysia report by Balakrishnan et al. (1999) provide a source of comparison for verifying the usefulness of the modulus reduction factor. The bored piles, five constructed in "dry" excavation and another five in "wet" excavation for which either water or bentonite was added during drilling, had similar dimensions: $L=15-40$ m; $d=600-750$ mm, and $L=10-40$ m; $d=600-1{,}200$ mm, respectively. Fitting parameters similar to those obtained from PMTs with borehole conditions simulating significant delay in concret-

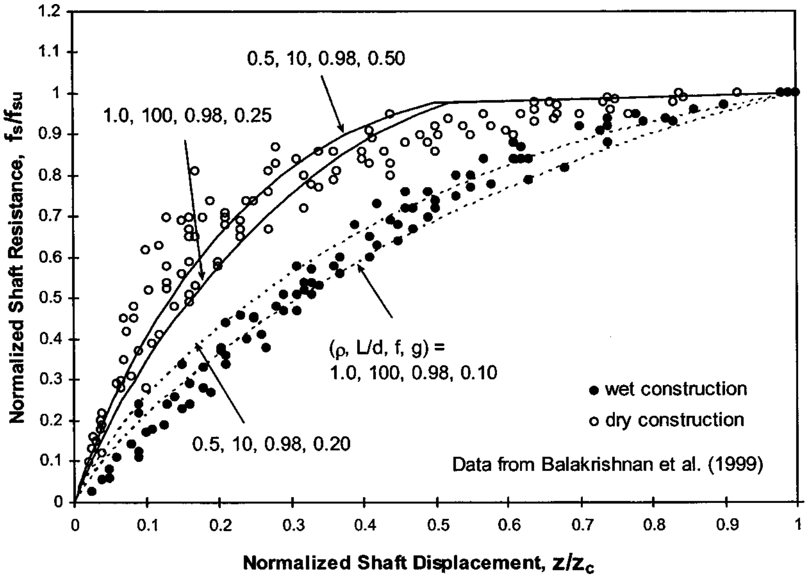

Fig. 11. Comparison of measured and predicted $t-z$ curves for bored piles in Kenny Hill Formation

ing in the Jurong Formation, which is geologically linked to the Malaysian rock Formation, were used in the analysis.

Fig. 11 shows a direct comparison of the predicted $t-z$ curves with the field load transfer curves reported by Balakrishnan et al. (1999). Note that $t-z$ curves in both "wet" and "dry" conditions have been normalized by the same z_c that corresponds to the "wet" condition, where z_c is the critical shaft displacement beyond which the shaft friction mobilizes or increases at a drastically reduced rate. The range of $t-z$ curves for "wet" construction calculated based on f of 0.98 and g of 0.1 for significant delay and 0.2 for moderate delay is seen to compare well with the field $t-z$ data for bored piles actually constructed in "wet" holes. The predicted $t-z$ curves for "dry" construction, which was generated by assuming that the g factor for "dry" condition was 2.5 times that for "wet" condition, also matches reasonably well with the measured $t-z$ data for piles actually constructed in "dry" holes. It is clear that using the fitting parameters f and g as obtained from PMTs, there is a good match between the predicted and the field $t-z$ curves for the bored piles.

Practical Applications

For general design applications, the modulus degradation fitting parameters f and g can be deduced from the pressuremeter test that incorporates unloading–reloading cycles in a freshly prepared borehole for bored piles that are expected to be constructed in "dry" condition without significant delay. Modulus fitting parameter g from triaxial compression tests on samples reconsolidated to in-situ mean effective stress should be reduced by half (based on Figs. 8 and 9), if PMT results are not available. Zhu and Chang's (2002) procedure can be used for the prediction of normalized $t-z$ curves. The limiting or critical shaft resistance, f_{su}, and the critical shaft displacement, z_c, that are required in the design can be estimated from correlations presented by Chang and Broms (1991) and Zhu and Chang (2002). A rational pile design can then be carried out based on the load transfer method using the predicted $t-z$ curves.

For bored piles that need to be constructed in the "wet" condition and a significant delay before concreting is likely, an appropriate modulus reduction factor f_G as obtained from Fig. 10 for a typical g_2/g_1 value of 2.5 can be applied so that the combined effect of stress relief and borehole soaking is properly reflected on the load transfer curves. A typical reduction of 20% can be applied to the limiting or the critical values of unit shaft resistance for bored piles for which the boreholes might be soaked for over 2 h or more during construction based on results from present study on the residual soils of the Jurong Formation in Singapore.

Conclusions

An investigation has been carried out on the NTU campus that is covered by the residual soil of the sedimentary Jurong Formation in Singapore to study the construction effects, particularly those lead to stress and moisture changes, on load transfer along bored piles. The main conclusions that can be drawn are as follows:

1. Laboratory moisture migration observations on compacted residual soil specimens in contact with cement mortar confirmed the dependency of moisture changes from wetting on the initial moisture content, the plasticity of the soil and the water/cement ratio in the mortar as observed by others.
2. Direct measurements of horizontal stress changes in the ground during the construction of a bored pile in compacted residual soil indicated a drastic reduction of horizontal stress after borehole excavation and a gradual recovery of the horizontal stress during and after concreting to a level of around 80 and 94% of the corresponding initial horizontal stresses, respectively, at 0.5 and 0.9 m away from the borehole wall.
3. Borehole shear tests and pressuremeter tests carried out in both "dry" and "wet" boreholes indicated a maximum reduction in measured friction angle of 20% and cohesion intercept of 80% after soaking of boreholes in water for 1–4 h. The observed reduction in the pressuremeter modulus typically ranged from 17 to 60% and in the net limit pressure from 20 to 60% after soaking of boreholes for 2–5 h, in comparison with tests in unsoaked boreholes.
4. The variation in stiffness with shear strain as observed in both the consolidated undrained triaxial compression tests

and the pressuremeter tests which simulated both ''dry'' and ''wet'' borehole conditions follows a hyperbolic function, although the fitting parameters are different between the two types of tests possibly due to differences in the simulated stress relief, mass of soil tested, and progressive failure.

5. With a proper simulation of the borehole conditions, the pressuremeter test which incorporates unload–reload cycles can yield representative modulus degradation curves that are suitable for the prediction of load transfer curves for bored piles constructed in ''dry'' or ''wet'' boreholes. The same approach should be applicable to bored piles that are constructed in other specific borehole conditions such as when bentonite slurry or polymer fluids are used in the stabilization.
6. A procedure has been developed to account for combined effect of stress relief and soaking of borehole in water in the prediction of relevant load transfer curves by introducing a modulus reduction factor. The recommended procedure is capable of capturing the influence of two extreme borehole conditions of ''dry'' and ''wet'' on the load transfer for bored piles constructed in the intermediate geomaterials in Singapore and Malaysia, and possibly other similar geological formations.

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Notation

The following notations are used in this paper:

c' = effective cohesion intercept;
d = diameter of shaft;
E_s = elastic modulus of soil;
f, g = fitting parameters for modified hyperbola;
f_G = modulus reduction factor;
f_s = unit friction;
f_{su} = ultimate unit shaft resistance;
G = shear modulus of soil;
G_{max} = maximum shear modulus of soil;
G_p = pressuremeter shear modulus;
I_d = material index from dilatometer test;
K = coefficient of earth pressure;
K_0 = coefficient of earth pressure at rest;
L = pile length;
N = standard penetration resistance (blow/0.3 m);
p_L = limit pressure from pressuremeter test;
p_L^* = net limit pressure from pressuremeter test;
p_0 = lift-off pressure in dilatometer test;
p_1 = pressure at 1 mm displacement of membrane at center in dilatometer test;
q_c = cone penetration resistance;
s_u = undrained shear strength;
w/c = water/cement ratio;
w_i = initial moisture content;
w_0 = natural moisture content;
z = shaft displacement;
z_c = critical shaft displacement;
ϵ_r = reference shear strain;
ϵ_s = shear strain;
ρ = inhomegeneity factor;
σ_c' = isotropic consolidation pressure in triaxial test;
σ_{v0} = in situ vertical total stress;
τ = shear stress;
τ_{max} = maximum shear stress; and
ϕ' = effective friction angle.

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