LOAD DEFORMATION ANALYSIS OF BORED PILES IN RESIDUAL WEATHERED FORMATION

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ABSTRACT: The current design practice of single bored piles in residual weathered formations is based mainly on stability consideration against shear failure, and pile deformation analysis is rarely carried out. However, the acceptance criteria of single piles during pile load tests during construction is based mainly on the permissible settlement criteria as specified in the specifications/codes. In this study, a reliable method of predicting the load deformation and load distribution curves is proposed for bored piles in a residual weathered formation (Kenny Hill Formation) in Kuala Lumpur based on: (1) considerations of the weathering profiles and the engineering characteristics of this formation; (2) field performance data of fully instrumented bored pile load tests; and (3) load transfer design characteristics of the load deformation behavior. In this deformation analysis, the pile installation methods and the nonlinear behavior of the pile material are incorporated. The proposed load deformation analysis was carried out on both the instrumented and noninstrumented piles, producing good results. Therefore, this proposed method can be used to predict the load deformation characteristics of single bored piles in weathered formation during the design stage.

INTRODUCTION

Bored and cast-in-situ piles, generally varying from 0.6 to 1.5 m in diameter, are extensively used as the foundation of heavy structures in Kuala Lumpur, Malaysia, because of their high bearing capacity, relatively low cost, easy length adjustments, low vibration, and noise levels during the installation. A large portion of these piles are constructed in residual weathered formation derived from clastic metasedimentary rocks (Kenny Hill Formation), which covers many parts of the city.

Currently, the widely used design practice of bored piles in this residual weathered formation follows the traditional approach. The ultimate shaft and base resistances are calculated using static formula, and are more often based on the empirical correlations using standard penetration tests (SPT) (Chiu and Perulmalswamy 1987; Toh et al. 1989). An overall factor of safety of 2.5–3.0 is normally applied to arrive at the working pile loads. In most cases, no load deformation analysis is carried out to determine the pile head deformations to ensure the serviceability compliance.

Confirmation of pile design is often verified using conventional pile load tests. However, in these tests, the confirmation of the design is generally based on the serviceability (settlement) compliance of the pile head against the permissible settlements as stipulated in the specifications or codes.

In the last ten years, a number of instrumented pile load tests have been carried out in the Kenny Hill Formation. Using the observed load transfer characteristics from these tests, and careful consideration of the geology and engineering characteristics of this residual weathered formation, a method of predicting the load deformation characteristics of single bored piles in this formation is developed. The load deformation analysis is carried out using the load transfer method and the field performance data of the instrumented piles. Nonlinear response of the formation and the pile material, as well as the effect of the construction methods, are also incorporated in this load deformation analysis.

The proposed load deformation analysis using the load transfer method was performed on both instrumented piles and noninstrumented piles, producing good results, validating the use of this method in predicting pile deformation characteristics during the design stage. Hence, with the load deformation results, a complete design of single bored piles in this weathered formation can be performed to satisfy both the ultimate (stability) and serviceability (settlement) requirements.

GEOLOGY: KENNY HILL FORMATION

The geological stratification of Kuala Lumpur is generally complex. The oldest rock is represented by the schist formation that is then followed by the Kuala Lumpur limestone. The Kenny Hill Formation, which was subsequently deposited, consists of a monotonous sequence of interbedded clastic sedimentary rocks such as sandstones, shales, and mudstones. This formation is also referred as metasedimentary, considering that the sedimentary rocks have been partly metamorphosed into quartzite and phyllite. The damp tropical climate has caused deep weathering profile in this formation. The weathered profile is often complicated due to the nonhomogeneity and textural variations of the parent rocks.

PREVIOUS STUDIES ON LOAD TRANSFER ANALYSIS OF PILES

The previous studies on the load transfer method of analysis of piles in residual or similar formations have been summarized in Table 1. In these studies, no considerations have been given to the method of pile construction, nonlinear response of pile material, and engineering behavior of ''soil-like'' and ''rock-like'' materials within the weathered formations.

INSTRUMENTED PILE LOAD TEST DATA

From eight sites within the Kenny Hill Formation in the Kuala Lumpur area, the results of thirteen numbers of instrumented bored pile load tests were used in this study.

Using the step integration method (Balakrishnan 1994), the instrumented piles results with both strain gauges and extensometers were analyzed to derive the load transfer curves for both shaft and base. In this analysis, the pile deformation computed using strain gauge readings at every instrumented level were also equated to the deformation computed using extensometer readings at the same levels. With this, the variation

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TABLE 1. Previous Studies on Load Transfer Analysis of Piles

Researcher (1)	Load transfer curves (2)	Soil formation (3)	Remarks (4)
Reese et al. (1969)	$f_s = f_{su}\{2(z_s/z_o)^{1/2} - z_s/z_o\}$ $z_o = 2 \times D \times \varepsilon$	Clay-shale Interbedded with sandstone	• 1 instrumented pile
Vijayvergiya (1977)	$f_s = f_{su}\{2(z_s/z_{su})^{1/2} - z_s/z_{su}\}$ $f_b = f_{bu}\{z_b/z_{bu}\}$	Clay-shale	• Analytical and empirical studies
Chang and Goh (1989)	Adopted Vijayvergiya's equations and established correlations for f_{su} (or f_{sc}) and z_{su} (or z_{sc}) with SPT N	Jurong Formation (weathered sedimentary) Singapore	• 9 experimental piles • Dry installation • 600 mm diameter
Chang and Broms (1991)	Adopted Vijayvergiya's equations and established correlations for f_{su} (or f_{sc}) and z_{su} (or z_{sc}) with SPT N	Jurong Formation (weathered sedimentary) Singapore	• 10 compression piles • From published results • Both experimental and actual piles

of pile deformation modulus, E_p with concrete stress levels in the piles, f_c were obtained. Using this relationship and the step integration procedures, the rest of the instrumented piles with only strain gauge readings were analyzed to derive the load transfer curves.

A critical point (f_{sc}, z_{sc}), known as load transfer parameters, was located on those load transfer curves with significant or full mobilization of shaft resistances. This point corresponds to either the maximum shaft resistance or a point on the load transfer curve whereby the slope is noticeably smaller after the point (showing strain hardening response). The shaft resistances and shaft displacements were normalized with the critical shaft resistances and critical shaft displacements, respectively, to obtain the normalized plots. The same procedure was adopted for the normalization of the base load transfer curves.

NORMALIZED LOAD TRANSFER CURVES

Pile Shaft

Two main factors that control the load transfer characteristics of piles are the method of pile installation and the soil properties. To incorporate the method of pile installation into the present analysis, the instrumented bored piles were classified into two broad classifications as follows:

1. Dry excavation: The piles were constructed in dry holes.
2. Wet excavation: The piles were constructed in wet holes stabilized using either water or bentonite during the drilling process with a short waiting period (<1 day).

The derived shaft load transfer curves for dry and wet excavation are shown in Figs. 1 and 2, respectively. Those curves were normalized using the shaft resistance (f_{sc}) and the critical shaft displacement (z_{sc}), which are summarized together with borehole details in Tables 2 and 3 for dry and wet excavation, respectively. The resultant normalized curves (f_s/f_{sc} versus z_s/z_{sc}) are shown in Fig. 3.

The load transfer characteristics for the two different pile installation methods are compared and the corresponding findings are summarized in Table 4. A comparison of the dry and wet normalized load transfer curves reveals the following:

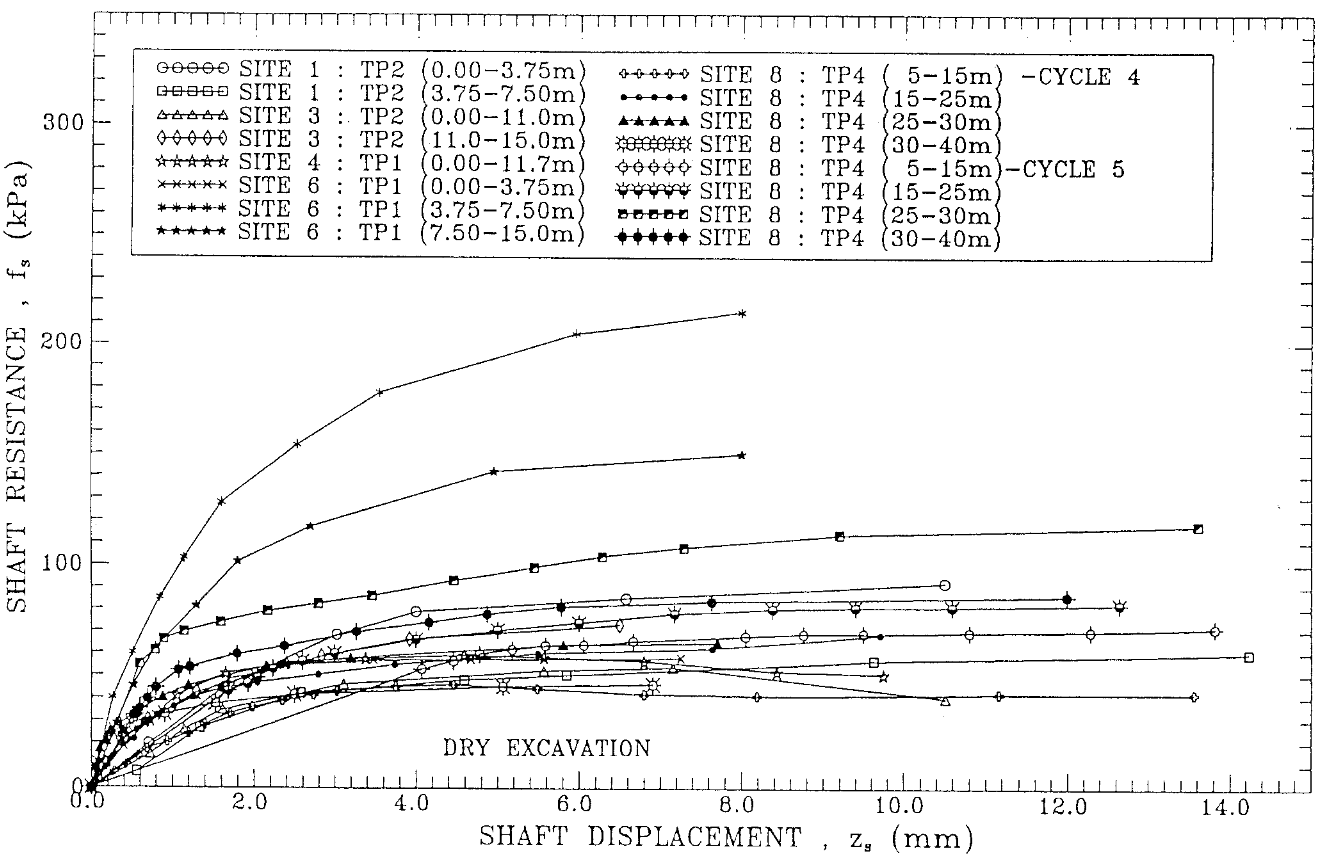

FIG. 1. Dry Excavation: Load Transfer Curves for Shaft

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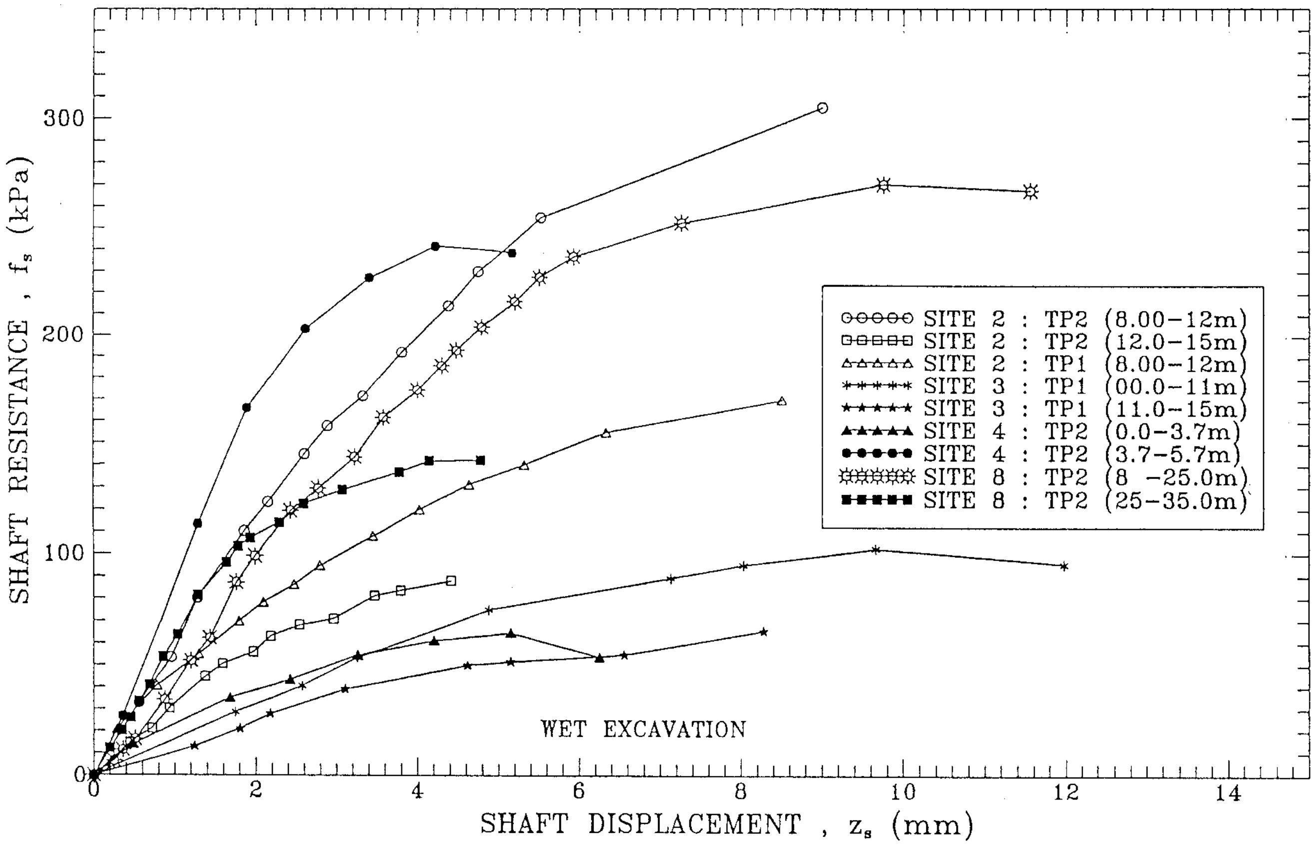

FIG. 2. Wet Excavation: Load Transfer Curves for Shaft

TABLE 2. Dry Excavation: Summary of Load Transfer Parameters for Pile Shaft

Site (1)	Test pile (2)	Pile diameter (mm) (3)	Soil stratum (4)	Depth (m) (5)	SPT *N* values (6)	Average SPT *N* (7)	f_{sc} (kPa) (8)	z_{sc} (9)
1	TP2	750	1. Stiff sandy silt	0–3.75	23	23	96	6.6
			2. Stiff sandy silt	3.75–7.5	7–17	11	56	4.6
			3. Hard sandy silt	7.5–15.0	26–68	60	105[a]	6.0
1	TP3	600	1. Very stiff sandy silt	0.3–3.75	11–30	20	102	—
			2. Hard sandy silt	3.75–7.5	34–54	44	133	—
			3. Very hard sandy silt	7.5–15.0	26–73	52	103	—
3	TP2	600	1. Very stiff clayey silt	0–11	16–22	20	53	7.2
				—	—	—	49	6.6
			2. Very stiff clayey silt	11–15	24–88	47	73	6.5
				—	—	—	> 98	>4
			3. Hard clayey silt	15–21	100–136	116	>138	>4.7
					—	—	>141	>2.1
4	TP1	685	1. Medium stiff sandy silt	0–11.7	8–29	15	59	4.6
			2. Hard sandy silt	11.7–15.5	116–133	125	—	—
5	TP1	760	1. Medium dense silty sand	0–14.3	8–33	18	>50	>1.0
				—	—	—	70[a]	4.0
			2. Weathered sandstone	14.3–15	300	300	>112	>0.4
				—	—	—	>251	>1.5
6	TP1	750	1. Very stiff clayey silt	0–3.75	6–64	35	>48	>2.2
				—	—	—	60	4.7
			2. Very dense silty sand	3.75–7.5	80–100	95	>101	>1.6
				—	—	—	215[a]	8.0
			3. Hard clayey silt	7.5–15	55	55	>64	>1.1
				—	—	—	150[a]	8.0
8	TP4	600	1. Very stiff clayey silt	5–15	11–26	22	46	4.5
				—	—	—	69	9.5
			3. Very stiff clayey silt	15–25	14–28	19	62	7.7
				—	—	—	82	12.6
			4. Hardy clayey silt	25–30	36–120	43	65	7.7
				—	—	—	118	13.6
			5. Very stiff clayey silt	30–40	28–44	36	46	5.0
				—	—	—	86	12.0

[a]Extrapolated values.

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TABLE 3. Wet Excavation: Summary of Load Transfer Parameters for Pile Shaft

Site (1)	Test pile (2)	Pile diameter (mm) (3)	Soil stratum (4)	Depth (m) (5)	SPT *N* values (6)	Average SPT *N* (7)	f_{sc} (kPa) (8)	z_{sc} (9)
2	TP1	1200	1. Hard sandy silt	8–12	22–100	61	170	8.5
			2. Hard sandy silt	12–16	107–125	116	270[a]	9.0
			3. Very dense silty sand	16–19.9	125–214	184	400[a]	10.0
2	TP2	1200	1. Hardy clayey silt	8–12	41–136	76	305	9.0
			2. Very dense silty sand	12–15	188–250	219	88	4.4
			3. Very dense silty sand	15–19	214–300	250	>424	>3.8
3	TP1 (Soft Base)	600	1. Very stiff clayey silt	0–11	16–22	20	102	9.7
			2. Very stiff clayey silt	11–15	24–88	47	@130	6.6
			3. Hard clayey silt	15–21	100–136	116	@215	<15
								7
4	TP2	840	1. Stiff sandy silt	0–3.7	—	45	65	5.2
			2. Hard sandy silt	3.7–5.7	—	100	242	8.0
			3. Hardy sandy silt	5.7–9.8	—	135	>427	>4.5
7	TP2 (Soft Base)	800	1. Very stiff clayey silt	0–21	9–23	18	72	—
				—	—	—	104	—
			2. Very dense silty sand	21–26	32–50	40	100	—
				—	—	—	214	—
			3. Hard clayey silt	26–30	50	50	88	—
				—	—	—	130	—
8	TP2	600	1. Hard clayey silt	8–25	78	78	>38	>2.1
				—	—	—	>91	−3.3
				—	—	—	>182	>6.2
				—	—	—	270	9.8
			2. Very hard clayey silt	25–35	100–150	115	>37	>0
				—	—	—	>24	>0.7
				—	—	—	>75	>4.0
				—	—	—	142	4.2
			3. Very hard clayey silt	35–40	300	300	>21	>0.1
				—	—	—	>8.8	>0.4
				—	—	—	>10.6	>3.9
				—	—	—	>38	>2.9

[a]Extrapolated values.

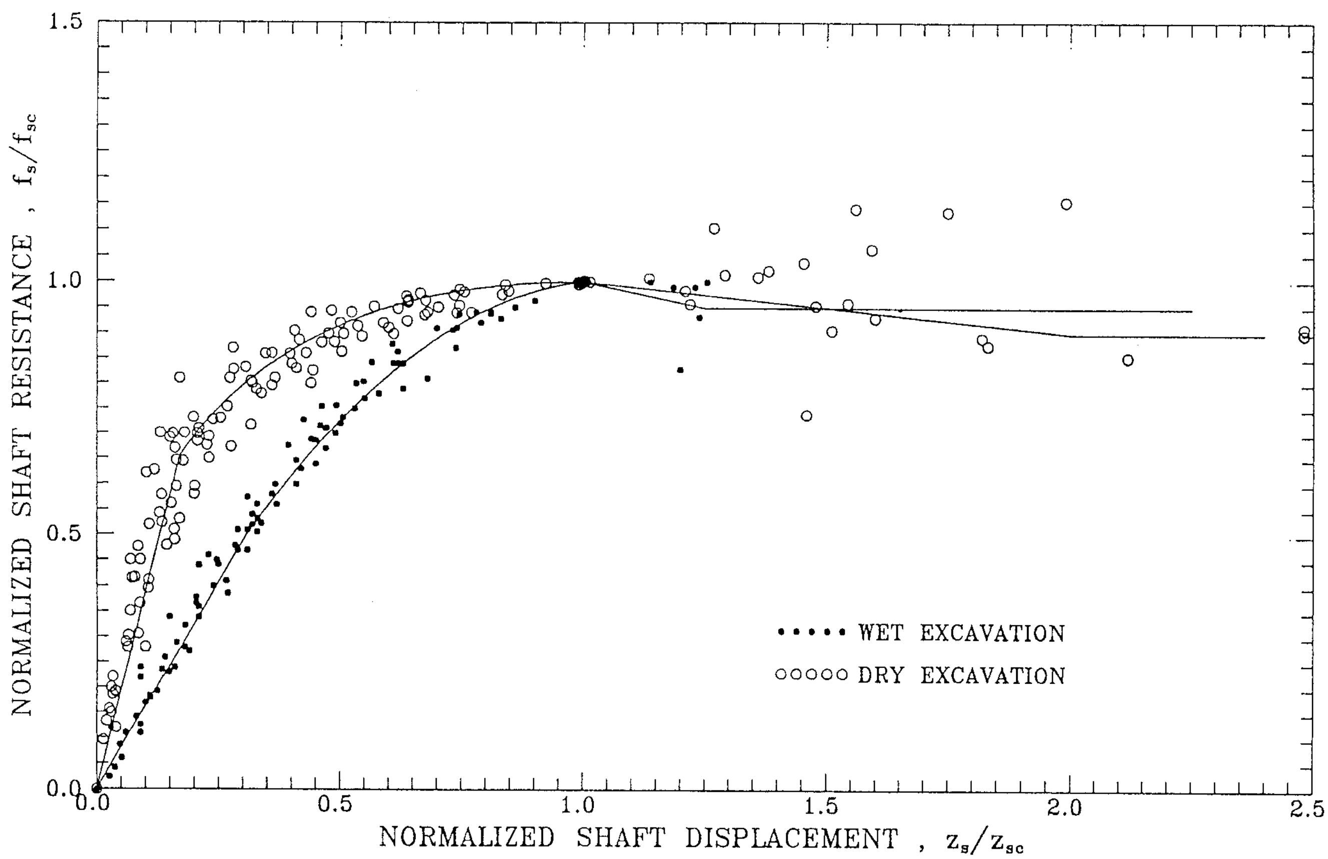

FIG. 3. Comparison of Load Transfer Curves between Dry and Wet Excavations

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TABLE 4. Shaft Load Transfer Characteristics for Dry and Wet Excavation

Dry excavation (1)	Wet excavation (2)
Normalized load transfer curve is unique with all points falling within narrow band.	Normalized load transfer curve is unique (better than that for dry excavation).
Load transfer curves generally show ductile response for soil-like materials (Grade VI to IV as per Balakrishnan 1994).	Similar to dry excavation.
Beyond critical values (f_{sc}), strain softening responses are mainly exhibited. Residual shaft resistance (f_{sr}) is 90% of maximum shaft resistance (f_{su}) corresponding to shaft displacement of 1.5–2.5 z_{sc}.	Strain softening response is exhibited in all cases. Instrumentation data are only available until 1.25 z_{sc}. It seemed that dilatancy property of weathered formation is reduced by softening effect of walls by water or bentonite. No definite conclusion on f_{sr}.
Normalized plot shows linear response until shaft displacement is 17% of the critical shaft displacement (z_{sc}). Beyond this, response is nonlinear.	Linear response until shaft displacement is 30% of z_{sc}. Transition from linear and nonlinear is not distinct, probably due to softening effect of borehole walls by water or bentonite.
Nonlinear response can be modeled quite accurately using relationship developed by Vijayvergiya (1977).	Nonlinear response cannot be modeled accurately using relationship developed by Vijayvergiya (1977).
Normalized load transfer curve is modeled using following equations:	Normalized load transfer curve is modeled using following equations:
For $z_s \leq 0.167z_{sc}$: Linear part	For $z_s \leq 0.31z_{sc}$: Linear part
$(f_s/f_{sc}) = 3.9(z_s/z_{sc})$	$(f_s/f_{sc}) = 1.61(z_s/z_{sc})$
For $0.167z_{sc} \leq z_{sc} \leq 1.0z_{sc}$: Nonlinear part	For $0.31z_{sc} \leq z_{sc} \leq 1.0z_{sc}$: Nonlinear part
$(f_s/f_{sc}) = 2(z_s/z_{sc})^{1/2} - (z_s/z_{sc})$	$(f_s/f_{sc}) = 1.9(z_s/z_{sc})^{1/2} - 0.9(z_s/z_{sc})$
For $1.0z_s \leq z_{sc} \leq 2.0z_{sc}$: Strain softening part	For $1.0z_s \leq z_{sc} \leq 1.25z_{sc}$: Strain softening part
$(f_s/f_{sc}) = 1.1(z_s/z_{sc})$	$(f_s/f_{sc}) = 1.2 - 0.2(z_s/z_{sc})$
For $z_s > 2.0z_{sc}$: Residual part	For $z_s \leq 1.25z_{sc}$: Residual part
$(f_s/f_{sc}) = 0.9$	$(f_s/f_{sc}) = 0.95$

- For load deformation analysis using the load transfer method, separate expressions are proposed to model the normalized load transfer curves for dry and wet excavation (Table 3).
- The proposed expressions to model the normalized load transfer curves reflect the response of the formation in the linear, nonlinear, strain softening, and the final residual zone.
- The fraction of the mobilized shaft resistance for a given fraction of shaft displacement for dry excavation is higher than that for wet excavation. This is probably due to the softening of the walls by water or bentonite in wet holes, particularly in the residual weathered formations that are strongly susceptible for such phenomenon.
- The $f_s/f_{sc}:z_s/z_{sc}$ ratio for the linear portion is related to the elastic stiffness of the soil, or more precisely to the coefficient of the vertical subgrade reaction. The ratio of this value between dry and wet excavation is about 2.4, which is very close to the value obtained using shear modulus back-analyzed from pile load tests (Balakrishnan 1994). This shows that the elastic stiffness of this residual weathered formation is reduced by ~50% in the wet holes.

Pile Base

Only five piles with some significant mobilization of the base resistance were considered in this analysis. All of them were constructed in a dry condition.

The derived base load transfer curves are shown in Fig. 4. The load transfer curves for Test Pile TP2 (Site 3) and TP1 (Site 4) showed some brittle response, because they correspond to average SPT N values >150 [the formation is classified as moderately weathered, Grade III or lower, as per the classification proposed by Balakrishnan (1994)].

The normalized base load transfer curve is shown in Fig. 5. The normalized plot proposed by Vijayvergiya (1977) represents the upper bound solution for the present data points. This

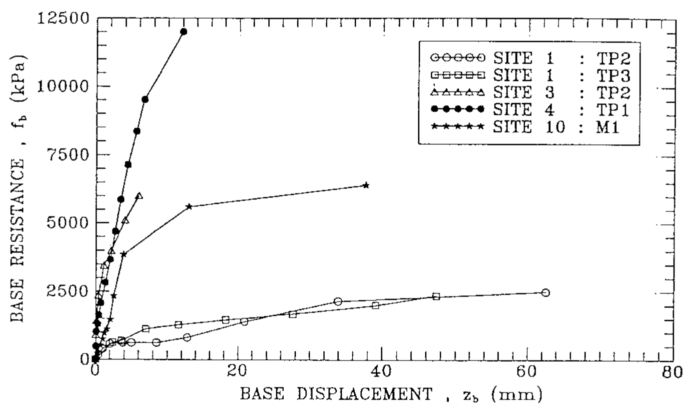

FIG. 4. Load Transfer Curves for Base

is probably due to the gradual softening that normally takes place at the base of the boreholes in residual weathered formations (they behave like an overconsolidated clay) as a result of the stress relief during drilling.

The normalized curve is not unique. The values fall within a broad band and a lot of scatter is noticed at low values of base displacements. This probably may be due to the limited data and a wide range of SPT N values at the base (from completely weathered to moderately weathered, which contribute to different engineering behavior).

Only a small amount of base resistance (<10% of the applied loads) is mobilized at design loads, and occasionally slightly higher base resistance (~30%) at twice the design loads for dry excavation. Whereas for wet holes, a negligible base load is mobilized even at twice the design loads (Balakrishnan 1994). Therefore, the contribution of the base resistance is not significant. Hence, any error in the modeling of the normalized load transfer curve will not significantly affect the predicted settlements until twice the design loads. Toh et al. (1989) has suggested that the base resistance be ignored in

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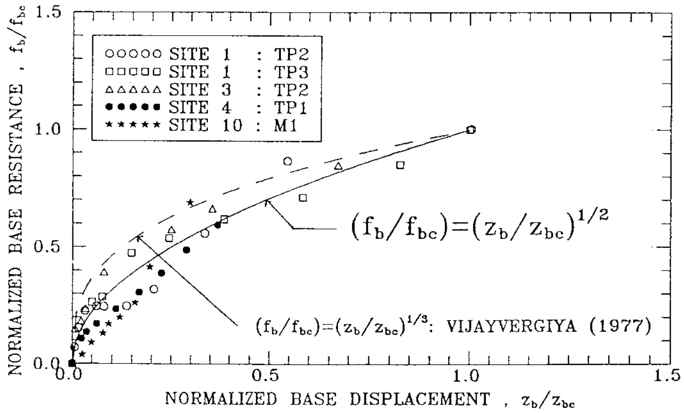

FIG. 5. Normalized Load Transfer Curve for Base

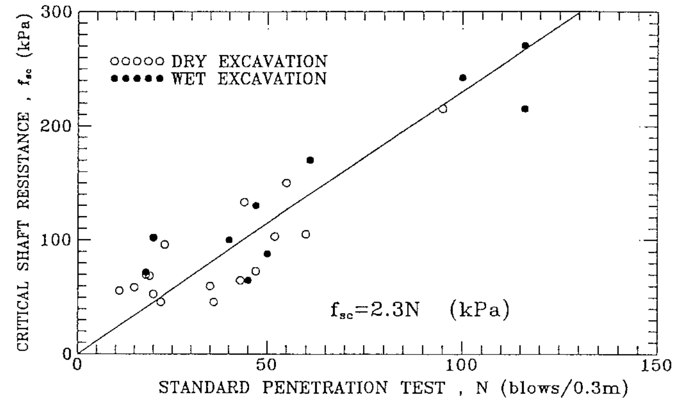

FIG. 6. Relationship between f_{sc} and SPT *N*

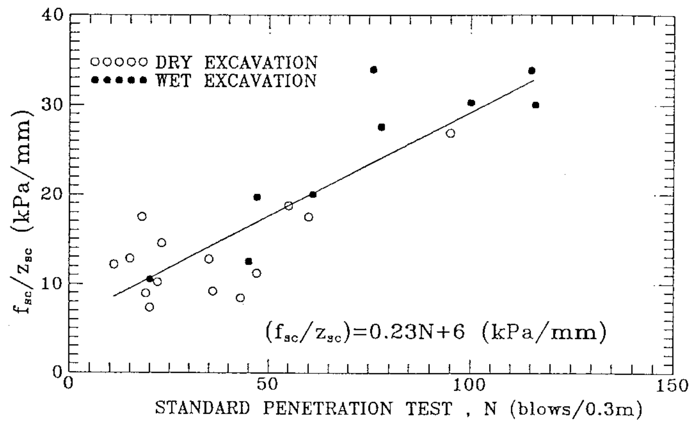

FIG. 7. Relationship between (f_{sc}/z_{sc}) and SPT *N*

determining the ultimate geotechnical capacity of bored piles in this formation.

For complete analysis of the load deformation behavior of the bored piles in Kenny Hill Formation, particularly those constructed in dry holes, the normalized load transfer curve for the base is approximately modeled using the following equation:

$$(f_b/f_{bc}) = (z_b/z_{bc})^{1/2} \tag{1}$$

LOAD TRANSFER PARAMETERS

Pile Shaft

The soil properties, the other factor that significantly affects the load transfer characteristics, were incorporated in the derivation of the load transfer parameters. SPT *N*, the only tests carried out consistently in this formation, can be used together with index properties to characterize the weathering profiles of this weathered formation (Balakrishnan 1994). Hence, in this study, SPT *N* values were used to correlate the load transfer parameters. As more sophisticated test results are available in the future, due to improvements in the soil investigation techniques, it will then be possible to determine better correlations.

The load transfer parameters seemed to be unaffected by the method of pile installation. This shows that the softening of the walls by water or bentonite only significantly affect the shape of the load transfer curves and not the peak or the critical values. A similar conclusion was quoted by Fleming and Sliwinski (1977) in a Construction Industry Research and Information Association publication that the use of bentonite suspension has no detrimental effect on the levels of the shaft resistance mobilized near the ultimate load condition. However, more data is required to make any definite conclusion on this matter.

All the load transfer parameters obtained from the load transfer curves of the instrumented bored piles correspond to SPT *N* values <150 that correspond to Grade VI–IV weathered rock materials. The behavior of moderately weathered rocks (Grade III and lower grades) are quite close to that of weak rocks and probably rocks (Balakrishnan 1994). Therefore, different relationships are derived for these two broad divisions (soil and weak rocks) as outlined in the following sections. The same normalized load transfer curve is assumed for both divisions due to unavailability of load transfer characteristics data for weak rocks.

(i) "Soil": Residual Soils (VI) to Highly Weathered Rocks (IV)

In a majority of the cases, the critical shaft resistance (f_{sc}) corresponds to the peak mobilized shaft resistance (f_{su}). The relationship obtained between f_{sc} and the SPT *N* values from this study (Fig. 6) is expressed as

$$f_{sc} = 2.3N \text{ (kPa)} \tag{2}$$

The relationship obtained from this study is in general agreement with the findings of Toh et al. (1989) and Chang and Broms (1991) for residual weathered formations.

In the majority of the cases, the critical shaft displacements (z_{sc}) corresponded to the shaft displacements at peak shaft resistances. The z_{sc} values obtained in this study of instrumented bored piles in Kenny Hill Formation ranged between 4 and 10 mm. The z_{sc} values are found to be generally independent of the pile diameter and soil properties. Similar findings were concluded by Chang and Broms (1991). Therefore, the ratio of (f_{sc}/z_{sc}) was selected for the correlation study, because this ratio represents the qualitative measure of the stiffness of the interface or more precisely related to the coefficient of vertical subgrade reaction. It has been shown by many researchers that the stiffness modulus is related to SPT *N* values. The correlation obtained from this study (Fig. 7) is given by

$$(f_{sc}/z_{sc}) = 0.23N + 6 \text{ (kPa/mm)} \tag{3}$$

(ii) "Weak Rocks": Moderately Weathered Rocks (III) and Lower Grades

From the collected and analyzed data of the instrumented piles in Kenny Hill Formation, there is no available information on the load transfer characteristics for this category. Table 4 summarizes some of the pile tests carried out on the moderately weathered Melbourne mudstone that is believed to be similar to the formation in this study. The data was compiled from the paper published by William and Pells (1981). The

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given unconfined compression strengths of the weathered formations were transformed to the equivalent SPT N values using Stroud's (1974) correlation for weak rocks ($s_u = 6N$ kPa). The computed SPT N values seemed to agree well with the classification proposed by Balakrishnan (1994) for the moderately weathered grade.

Radhakrishnan and Leung (1989) have found that the peak shaft resistance for the weathered rocks in Jurong Formation agree closely to those predicted using the Rowe and Armitage (1987) relation, $f_{su} = 0.45(q_u)^{1/2}$ (MPa), where q_u is the unconfined compressive strength of rocks. Using Stroud's (1974) correlation, this equation is transformed to $f_{su} = 49(N)^{1/2}$ (kPa).

Using this relation, the ratio of $f_{su}/N^{1/2}$ was computed for the data in Table 4 and the values are found to be in close agreement to the above equation. To allow for potential variability in rock properties with both position and time, Rowe and Armitage (1987) have suggested a reduction factor of 0.7 to be applied. Adopting this recommendation, the critical shaft resistance used in the load transfer analysis is given by

$$f_{sc} = 35(N)^{1/2} \text{ (kPa)} \quad (4)$$

Little information is available in published literature with regard to the critical shaft displacement. Horvath and Kenny (1979) found that the shaft resistance generally becomes fully mobilized at a relatively small displacement, ~6 mm. From the correlation obtained in this study for ''soil-like'' material, the maximum critical shaft displacement is 8 mm. Therefore, a constant critical displacement of 8 mm is assumed for weak rock in the load transfer analysis.

Pile Base

Only two instrumented piles were tested to failure. The results of these two piles are tabulated in Table 5. The critical base displacement, z_{bc} is found to be a function of the pile diameter. For the two piles in this study (diameter of <1.0 m) the value is ~6–8% of pile diameter. Similarly, Vijayvergiya (1977) has observed the z_{bc} to be between 3 and 9% of the pile diameter for piles, both in clays and sands.

The (f_{bc}/N) value of 40–43 obtained from the instrumented piles were close to the value of 40 suggested by Chang and Broms (1991) for a pile diameter <1.0 m. Chiu and Perumalswamy (1987) have suggested a value of 50 for Kenny Hill Formation.

From the results, it can be seen that a large base displacement is required to fully mobilize the base resistance. Therefore, at design load, the corresponding load transfer parameters, f_{bc} and z_{bc}, do not significantly affect the prediction of head settlement. However, if the base resistance is to be considered in the load transfer analysis, particularly for dry holes, then the following equations are proposed:

TABLE 5. Data on Test Piles in Moderately Weathered Melbourne Mudstone (Williams and Pells 1981)

Reference (1)	Site (2)	Test pile (3)	q_a (kPa) (4)	Equivalent SPT N (5)	f_{su} (kPa) (6)	$f_{su}/\sqrt{N}$ (kPa) (7)
Johnston and Donald (1979)	Flinders Street	F1	3,060	255	1,050	65
		F2	1,930	160	940	74
William (1980)	Middleborough Road	M1	2,460	205	600	41
		M2	2,300	190	640	46
		M3	2,300	190	710	51
		M4	2,340	195	620	44
William (1980)	Westgate Fory	WG303/2	3,490	290	890	49

Note: q_u = Unconfined compressive strength = $2s_u$; $s_u = 6N$ kPa (Stroud correlation).

For pile diameter <1.0 m

$$f_{bc} = 40N \text{ (kPa) and } z_{bc} = 8\% \text{ of pile diameter} \quad (5)$$

For pile diameter >1.0 m

$$f_{bc} = 5s_u = 30N \text{ (kPa) and } z_{bc} = 10\% \text{ of pile diameter} \quad (6)$$

Since no test results are available, the recommendations suggested by Chang and Broms (1991) are used in the load transfer analyses.

PILE DEFORMATION MODULUS

In this study, an indirect approach was adopted as described previously to determine the variation of the in-situ pile deformation modulus with the stress levels in the piles using the rod extensometer and strain gauge readings. The stress here refers to the average stress over a horizontal pile cross section. The results obtained from this analysis are plotted in Fig. 8. All the piles used in this analysis are lightly or nominally reinforced (0.3–1.0% steel).

The pile deformation modulus is found to be a function of the applied load. A similar conclusion was found by Chan (1975) for piles in the same formation. In the virgin loading stage, the pile deformation modulus decreases linearly with the increase in the stress levels in the pile. The highest stress level recorded was ~$0.7 f_{cu}$, where f_{cu} is the grade or cube strength of the concrete used. This value corresponds to an applied load of about three times the design load. The linear variation of the pile deformation modulus in the virgin loading is almost parallel for all the three concrete grades used in this study.

All the piles with concrete grade of 30 MPa were subjected to virgin load tests. The initial values of the pile deformation modulus correspond closely to the values of the elastic moduli of concrete (E_c) normally obtained using the equation, $E_c = 9.1 f_{cu}^{0.33}$ (kN/mm^2), where f_{cu} is in MPa as per British Standard BS 8100. The variations in the E_p values were ~25% from zero stress to about $0.4 f_{cu}$ (which corresponds to applied load of two times the design load). This variation conformed well to the findings of Chan (1975) of ~29–30% in any particular load test.

Such significant variation in the pile deformation modulus is probably due to: (1) The piles are lightly reinforced; (2) segregation of concrete during placing; and (3) lateral confinement provided by the stiff residual weathered formations.

The nonlinear behavior of the pile material has to be considered in the prediction of load settlement and load distribution curves in the piles using the load transfer method, particularly for long piles. From the limited data available from this study, the following equation is suggested to forecast the pile deformation modulus with variation in the stress values in the pile in a virgin loading cycle:

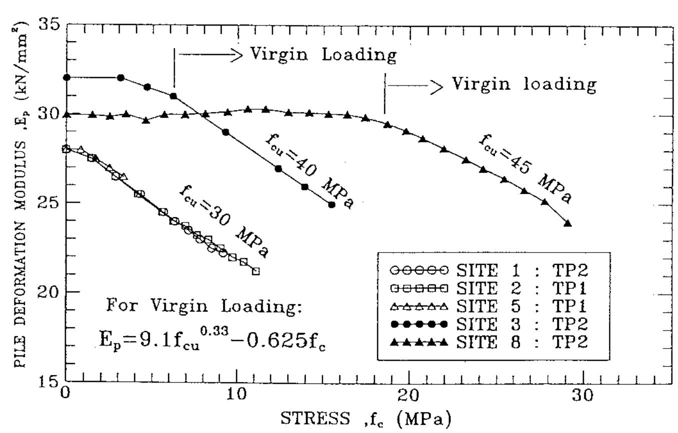

FIG. 8. Variation of Pile Deformation Modulus with Stress

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$$E_p = 9.1 f_{cu}^{0.33} - 0.625 f_c \text{ (kN/mm}^2\text{)} \quad (7)$$

where f_{cu} and f_c are in MPa.

LOAD DEFORMATION ANALYSIS

A computer program (LTRANB) was developed using the load transfer formulation for the synthesis of the load settlement and load distribution curves of single bored piles in the Kenny Hill Formation. The load transfer method was first proposed by Seed and Reese (1957) and subsequently extended by Coyle and Reese (1966). All the findings from the field performance data of the instrumented piles in the same formation were incorporated in this program (Table 6).

In this program, the pile is idealized as a series of elastic nonlinear discrete elements interacting with elastic nonlinear spring elements above and below, and with a side nonlinear spring and a base nonlinear spring that represents the soils. The side soil springs are assumed to be fixed at their outer extremity, thus making each soil layer independent of all other layers. The normalized load transfer curves coupled with a method of estimating the load transfer parameters are used to describe the load deformation properties of the side and base soil springs. The relationship between the variation in the pile deformation modulus (assumed to be the same as the concrete elastic modulus) with stress level is used to describe the nonlinear pile spring element.

The synthesis of load settlement and load distribution curves using the developed program was first carried out on the instrumented bored piles (Site 1 to Site 8). The contribution of the base resistance was only considered in the dry excavation. Typical results obtained using this program were compared with the measured curves as shown in Figs. 9 and 10. The predicted curves agreed closely with the measured curves in the majority of the cases. Slight deviations are observed for the load settlement curves beyond twice the design load. Deviations are also noticed on the load distribution curves close to the base. This is probably due to the conservative approach adopted for the base resistance characteristics for lack of sufficient data. These results generally confirmed the findings of this study on the load transfer characteristics.

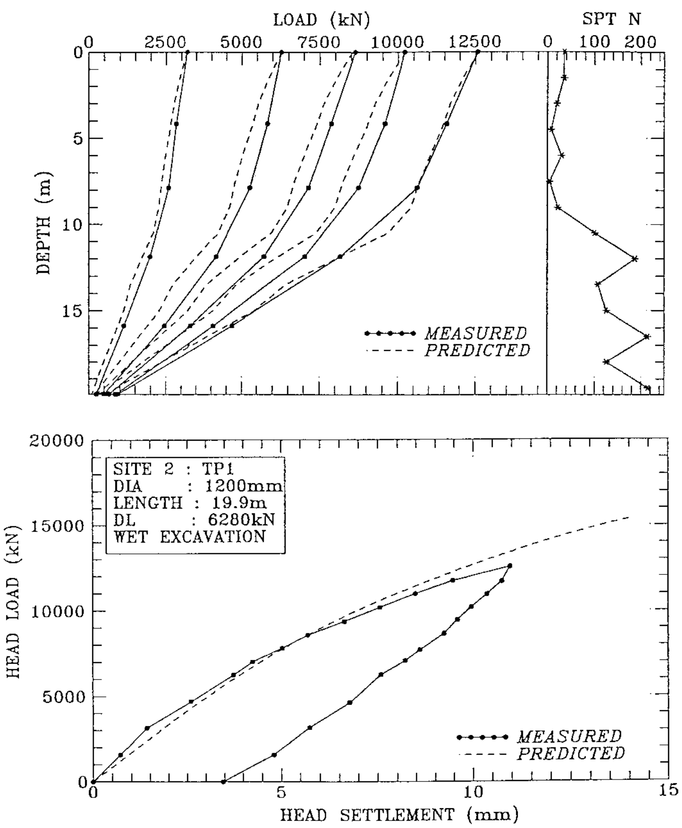

FIG. 9. Predicted versus Measured Load Settlement and Load Distribution Curves for Instrumented Bored Pile TP1 at Site 2

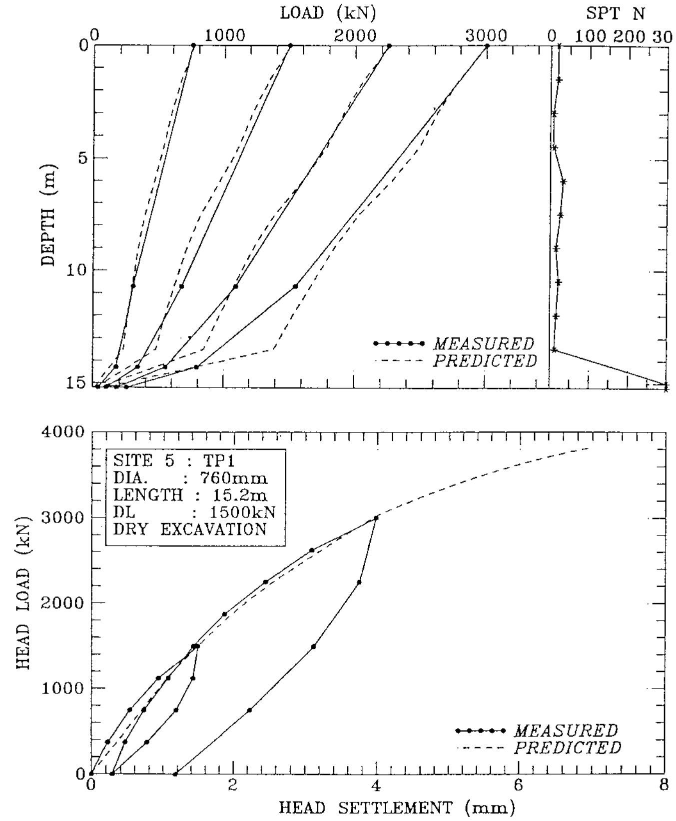

FIG. 10. Predicted versus Load Settlement and Load Distribution Curves for Instrumented Bored Pile TP1 at Site 5

TABLE 6. Load Transfer Parameters for Pile Base

Site (1)	Pile (2)	Diameter (mm) (3)	f_{bc} (kPa) (4)	z_{bc} (mm) (5)	N (6)	f_{bc}/N (kPa) (7)	z_{bc}/Diameter (%) (8)
1	TP2	750	2,490	62.5	62	40	8.3
10	M1	200	5,602	13.0	130	43	6.5

The program was further tested on noninstrumented bored piles in the same formation (Site A to Site E) to verify the applicability and versatility of this program to general cases. Similar results were obtained whereby the predicted curves were generally in good agreement with the measured ones.

A comparison of the predicted and measured head settlements at design load and twice the design loads for all the bored piles analyzed using this program are shown in Table 7 (dry excavation) and Table 8 (wet excavation). It can be seen that the developed program using the load transfer method and the field performance data in the Kenny Hill Formation produced good results in predicting head settlements of bored piles at design and twice design loads in the same formation.

SUMMARY AND CONCLUSIONS

The current design practice of single bored piles in the weathered metasedimentary formation (Kenny Hill Formation) in Kuala Lumpur is based mainly on stability consideration against shear failure. In a majority of the cases, the load deformation analysis is not carried out to predict the settlement of piles at working and twice working loads, although the

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TABLE 7. Dry Excavation: Comparison of Measured and Predicted Head Settlements

Site (1)	Test pile (2)	Pile diameter *D* (mm) (3)	Pile length *L* (m) (4)	Design load (kN) (5)	At Design Load Measured (mm) (6)	At Design Load Predicted (mm) (7)	At 2X Design Load Measured (mm) (8)	At 2X Design Load Predicted (mm) (9)
1	TP2	750	15.0	2,500	5.60	4.10	—	—
3	TP2	600	21.0	1,750	2.74	2.59	7.72	7.22
4	TP1	685	15.5	2,000	1.54	2.32	6.01	6.30
5	TP1	760	15.2	1,500	1.49	1.50	3.98	3.94
6	TP1	750	15.0	2,400	2.60	1.95	9.10	8.66
8	TP4	600	40.0	1,500	2.21	2.30	7.20	6.61
A	TP1	1,450	30.0	9,400	4.65	4.60	15.00	14.95
A	TP2	600	29.0	1,750	2.41	3.05	8.12	10.56
C	TP1	600	40.0	2,300	3.72	3.78	13.00	11.60
E	TP1	910	19.0	4,800	3.30	2.74	10.69	12.55

TABLE 8. Wet Excavation: Comparison of Measured and Predicted Head Settlements

Site (1)	Test pile (2)	Pile diameter *D* (mm) (3)	Pile length *L* (m) (4)	Design load (kN) (5)	At Design Load Measured (mm) (6)	At Design Load Predicted (mm) (7)	At 2X Design Load Measured (mm) (8)	At 2X Design Load Predicted (mm) (9)
2	TP1	1,200	19.9	6,280	3.74	3.93	10.60	10.00
2	TP2	1,200	18.9	6,280	3.68	4.00	9.73	9.80
3	TP1	600	21.0	1,750	4.19	3.47	9.22	8.72
4	TP2	840	9.8	3,000	2.73	3.07	6.83	7.44
7	TP2	800	30.0	3,140	4.72	4.94	—	—
8	TP2	600	40.0	2,300	4.26	4.02	10.10	10.22
B	TP1	1,140	15.0	6,370	4.75	6.20	14.50	14.20
C	TP2	600	40.0	1,500	3.77	3.00	8.50	7.00
D	TP1	1,500	49.0	11,000	12.00	10.00	—	24.00

acceptance criteria of bored piles during the pile load tests is based only on the permissible settlements stipulated in the specification or codes.

In this study, an attempt has been made to predict the load deformation characteristics of the piles in this formation using the load transfer method and the field performance data of the previous fully instrumented bored piles in the same formation. In the process of formulating the load transfer method of analysis of bored piles for settlement predictions, the following conclusions are drawn:

1. The normalized load transfer curves for the shaft are significantly influenced by pile installation methods and are unique for each case. Separate curves are proposed for dry excavation and wet excavation of bored piles with a short waiting period. The expressions proposed to model the normalized load transfer curves in the linear, nonlinear, strain softening, and the residual portions are tabulated in Table 4.
2. If the contribution of the base resistance is considered in the load transfer analysis, particularly for dry excavation, then the normalized load transfer curve for the base can be modeled using (1).
3. The correlations of the shaft load transfer parameters with SPT *N* value are proposed [(2)–(4)] for two broad classifications of the weathering profiles (soil and weak rocks materials).
4. The correlations of the base load transfer parameters with SPT *N* values are also proposed for two broad categories of pile diameters [(5) and (6)].
5. The variation of pile deformation modulus with concrete stress is quite significant in the lightly or nominally reinforced piles. This nonlinear behavior of the pile material needs to be considered in the load transfer analysis, particularly for long piles. A relationship is proposed (7) to determine the pile deformation modulus at various concrete stress levels.
6. The developed load transfer program (LTRANB) incorporating all of the above findings from the field performance data predicted quite accurately the load settlement and load distribution curves of the instrumented and noninstrumented piles in this formation.

Therefore, using this method, the load deformation and load distribution curves of bored piles in this formation can be predicted reasonably well during the design stage. With this, a complete design of single bored piles in Kenny Hill Formation can be carried out considering both the ultimate state (stability against shear failure) and the serviceability state (permissible settlements) requirements.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

D = pile diameter;
E_c = elastic modulus of concrete;
E_p = pile deformation modulus;
E_s = Young's modulus of soil;
FS = factor of safety;
f_b = base resistance;
f_{bc} = critical base resistance;
f_{bu} = maximum or peak base resistance;
f_c = compressive stress in pile;
f_{cu} = concrete strength or grade;
f_s = shaft resistance;
f_{sc} = critical shaft resistance;
f_{sr} = residual shaft resistance;
f_{su} = maximum or peak shaft resistance;
SPT N = standard penetration test (blows/300 mm);
s_u = undrained shear strength;
z_b = base displacement;
z_{bc} = critical base displacement;
z_s = shaft displacement;
z_{sc} = critical shaft displacement;
z_{su} = shaft displacement corresponding to f_{su}; and
ε = average failure strain (in %) from unconfined compression tests on soil sample.

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