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PREDICTIONS OF CAPACITIES OF FOUR TEST PILES AT NORTHWESTERN UNIVERSITY

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ABSTRACT: This paper presents the predictions of the axial load distributions of 4 test piles at approximately 2 weeks, 1 month and 1 year after installation. Evaluations of residual stress, pore pressure dissipation and other relevant factors were considered in the predictions, but are not explicitly presented in this paper. The shaft resistance in sand is predicted based on the LCPC method. A new approach, the Kc method, is proposed in this paper to estimate the shaft resistance in clay. Engineering judgment was exercised to take into account the installation influences and mobilization effects.

INTRODUCTION

The axial load distributions of 4 test piles at approximately 2 weeks, 1 month, and year after installation are presented in this paper. The test site is on the campus of Northwestern University, Evanston, Illinois. All these test piles are 50 ft (15.24 m) long, penetrating through about 23 ft (7.01 m) of dense sand overlying medium stiff clay. Two tested driven piles include a close-ended steel pipe pile and an HP 14x73 pile. Two drilled concrete piers were installed after all the other piles were driven, one by the slurry method and the other cased to 32 ft (9.75 m).

SUBSOIL CONDITIONS

According to the given data package, the soil deposits at the site consist of the following materials in a descending order: 23 ft (7.01 m) of fine grained sand (SP), 45 ft (13.72 m) of soft to medium clay (CL), 12 ft (3.66 m) of stiff clay (CL), and 10 ft (3.05 m) of hard silt (ML). Underlying the soil deposits is Niagaran dolomite.

The sand deposit at site is a fill that was placed in 1966 without special densification. However, field test results indicate

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that the sand is medium dense to very dense. Menard PMT results also show that OCR is 3.5 and 3.8 where tested. This suggests that the sand is overconsolidated. A careful inspection of the investigation results indicates that the top 23 ft (7.01 m) of sand can be further divided into 3 strata. The compactness is generally dense to very dense for the top 5 ft (1.52 m), medium dense for the middle 7.5 ft (2.29 m), and very dense for the lower 10.5 ft (3.20 m). Based on the available information, engineering judgment was exercised to obtain the range of SPT blow counts and the estimated friction angles of each stratum. Summary of sand properties is presented in Table 1 for reference.

The geology of the site and the low liquidity index indicate that the clay deposits, are overconsolidated. Marchetti's dilatometer test (DMT) results show that the OCR of the clay varies between 1.2 and 1.8 between the depths of 25 ft (7.62 m) and 49 ft (14.94 m). Though the oedometer test results show that some samples are normally consolidated to underconsolidated, further review of the compression curves of these oedometer test results indicates that these samples may be subject to severe disturbances and thus the maximum past pressure may have been underestimated. It can be concluded that the clay deposits are overconsolidated and the maximum past pressure is estimated to be 0.6 tsf (57.5 kN/m^2), larger than the effective overburden pressure. The variation of OCR with depth is presented in Table 2 along with other physical and engineering properties of the clay deposits between 23 ft (7.01 m) and 68 ft (20.73 m). The coefficient of earth pressure at rest, Ko, is obtained mainly by the DMT results and is expressed as a function of OCR. Since detailed results of Ko-consolidated triaxial tests are not available, it is suspected that the implied Ko value of 0.5 may not be representative. The Piezocone test results are not satisfactory as the dissipation time is too short to reliably evaluate the coefficient of consolidation. The vertical coefficient of consolidation in the normally consolidated range can be estimated from oedometer test results which are fairly consistent and are in a range of between $2x10^{-7}$ to $3x10^{-7}$ m^2/sec. The strength data obtained from various tests scatter significantly. The Ko-consolidated triaxial tests only reconsolidated the samples to their in-situ effective overburden pressure which is believed to be not large enough to overcome the disturbance effect. Based on the interpretation of all available strength data, the relationship

Table 1 Summary of Sand Properties

Depth, ft	SPT N-value	Ø
0 - 5	18-64 (avg.=46)	39^{o}
5 - 12.5	29-49 (avg.=36)	32^{o}
12.5- 23	52-72 (avg.=62)	40^{o}

+ 1 ft = 0.3048 m

Table 2 Summary of Clay Properties

Index properties	W_N (%) = 22.8 ± 2.1 W_L (%) = 37.8 ± 2.8 W_p (%) = 19.3 ± 3.6 LI (%) = 20.3 ± 2.1 PI (%) = 18.6 ± 2.2
Stress history (depths between 23 ft+ and 50 ft)	$\bar{\sigma}_{vo}$ (psf) = 1834 + 58 (Z-23), Z in ft $\bar{\sigma}_{vm}$ (psf) = $\bar{\sigma}_{vo}$ + 1200 Ko = 0.55 , OCR = $\bar{\sigma}_{vm} / \bar{\sigma}_{vo}$
Consolidation properties (depths between 23 ft and 50 ft)	c_h (OC) = 2.5 x 10^{-6} m^2/sec c_h (NC) = 2.5 x 10^{-7} m^2/sec
Strength properties.	Triaxial Compression: $s_u / \bar{\sigma}_{vo} = 0.33\ (OCR)^{0.8}$, $\emptyset = 30^o$ Triaxial Extension: $s_u / \bar{\sigma}_{vo} = 0.23\ (OCR)^{0.8}$, $\emptyset = 35^o$

\+ 1 ft = 0.3048 m
++ 1 psf = 0.0479 kN/m^2

between OCR and the strength ratios of triaxial compression and extension tests are presented in Table 2.

No long-term groundwater monitoring results are given. The analysis was carried out based on the assumption that the groundwater table is 10 ft (3.05 m) below ground surface and the groundwater pressure is in a hydrostatic condition.

SHAFT RESISTANCE IN SAND

In general, the shaft resistance in sand is estimated by the Laboratoire Central des Ponts et Chaussees (LCPC) method (5) with the estimation of the skin friction per unit area, fs generally expressed as follows:

$$f_s = q_c / \alpha \leq q_\ell \tag{1}$$

in which q_c is the tip resistance obtained from CPT, α is the coefficient depending on the pile type and soil condition, and q_ℓ is its corresponding limiting value. The values of α and q_ℓ used in this study are 200 and 120 kN/m^2 for driven piles, and 300 and 80 kN/m^2 for bored piers, respectively. Other analyses were also conducted, but only used as reference. Comparisons indicate that the results predicted by Kulhawy's method (7) are in fairly good agreement with those of the LCPC method if Ko of 1.5 is used.

Generally speaking, the shaft resistance in sand is considered to be independent of time. However, some special considerations should be applied to each type of test pile. These factors are discussed as follows:

(1) It is noted that a 19-in (0.48 m) boot plate is placed at the bottom of the steel pipe pile. During pile driving, this plate is expected to squeeze out the sand in the immediate vicinity of the pile shaft and loosen the dense sand adjacent to the pile shaft. Afterwards, the soil in the vicinity of the pile will again become denser due to the shearing during the pile load tests. The situation is quite complicated, and the reduction factor can only be determined based on the authors' experiences. Factors of 0.8, 0.85 and 0.9 are applied in predicting the pile capacity at 2 weeks, 1 month, and 1 year of pile load test, respectively.

(2) The H pile was driven with its instruments protected by one C8x18.75 section on each side of the web. There was a plate welded flush to bottom of the C sections. Soil plugging is expected, but the effect of plugging cannot be easily determined. In the current analysis, the area of contact with soil of the H pile is assumed to be equivalent to a rectangular pile with dimensions of the flange and depth of the HP 14x73 section. A reduction factor of 0.9 was applied in the calculation of the skin friction.

(3) The slurry pier was installed by using a 24-in (0.61 m) diameter casing with a length of 10 ft (3.05 m). The diameter of the pier below the casing is 18 in (0.46 m). Therefore, the contribution in capacity of the annular area should be considered. In the calculation, it is assumed that a pile length of 2.5 ft (0.76 m) below the casing depth will behave like a tapered pile shaft with unit skin friction 2 times that of a vertical shaft.

(4) The cased pier was installed by using a 24-in (0.61 m) diameter casing to a depth of 32 ft (9.75 m) penetrating through the sand deposits to the clay layer. No special consideration is made for the calculation of its friction resistance in sand.

SHAFT RESISTANCE IN CLAY

A recent study conducted at MIT has concluded that the horizontal effective stress acting on the pile shaft after consolidation is the pivotal factor affecting the skin friction of piles in slightly overconsolidated clays (1). Instead of using the conventional β method ($fs = \beta \times \bar{\sigma}_{vo}$), they expressed fs in terms of two new parameters as follows:

$$fs = \rho \times K_c \times \bar{\sigma}_{vo} \tag{2}$$

$$\rho = fs / \bar{\sigma}_{hc} \tag{3}$$

$$Kc = \bar{\sigma}_{hc} / \bar{\sigma}_{ho} \tag{4}$$

in which $\bar{\sigma}_{hc}$ is the horizontal effective stress acting on the pile shaft after consolidation. In practice, parameters ρ and Kc can either be predicted by analytical methods or measured directly by in-situ tests which can be regarded as model piles. Some encouraging results have been obtained although extensive studies following this approach have only been conducted in Boston Blue Clay and Empire Clay in Louisiana (2). Since analytical and measurement tools following this approach are not available, previous research results have been used as major references to implement this approach. Table 3 presents the physical properties of Boston Blue Clay, Empire Clay and Chicago Clay at site. A comparison of the physical properties implies that the results gained from Boston Blue Clay are more suitable to the project site than those of the Empire Clay. Certain modifications were made in this study so that this approach can be applied without utilizing very sophisticated analytical tools and advanced in-situ tests. The proposed method is named as "the Kc method" in order to distinguish this simplified approach from the MIT method which was built upon a sound theoretical base and verified by field measurements.

Figure 1 shows the normalized measurements of horizontal effective stress on pile shaft during soil consolidation following installation in lower Boston Blue Clay (2). After pile driving, field measurements indicate that Ko = Kc at the time that the excess pore pressure are fully dissipated. That Kc will equal Ko after consolidation constitutes one of the most important assumptions used in this study. In order to take the plugging effect into account, a parameter, p, was introduced (1) to normalize the pore pressure dissipation curve. The degree of plugging, p, is defined as the ratio of the displaced soil volume (outside the pile walls), As, to the soil volume ahead of the outer pile perimeter, i.e.,

$$P = As / (\pi \times B \times B / 4) \tag{5}$$

in which B is the outer diameter of the pile. The parameter p can express the ratio of the displaced soil due to pile installation for different pile types and can thus treat various piles in a consistent manner. Based on a simplified approach, this study proposes a normalized set-up curve (Fig. 2), transformed from Fig. 1 by assuming the horizontal coefficient of consolidation in OC range of lower Boston Blue Clay is 4×10^{-6} m^2/sec (4) and Kc = Ko at 100% consolidation after a close-ended pile (p=1) is driven. Figure 2 is used in this study to estimate the set-up rate for different piles with various degrees of soil displacement caused by pile installation.

The estimation of parameter ρ is also a very challenging task. Based on a very limited data base, it is noted that ρ value for a clay deposit appears to be close to the strength ratio ($\tau h/ \bar{\sigma} vc$) obtained from the direct simple shear test on a normally

Table 3 Comparisons of Physical Properties of Clay Deposits at Site, Boston Blue Clay and Empire Clay

	Clay Properties at Site	Boston Blue Clay (6)	Empire Clay (1)
W_L (%)	38±3	42±3	87±7
PI (%)	19±2	21±3	59±7

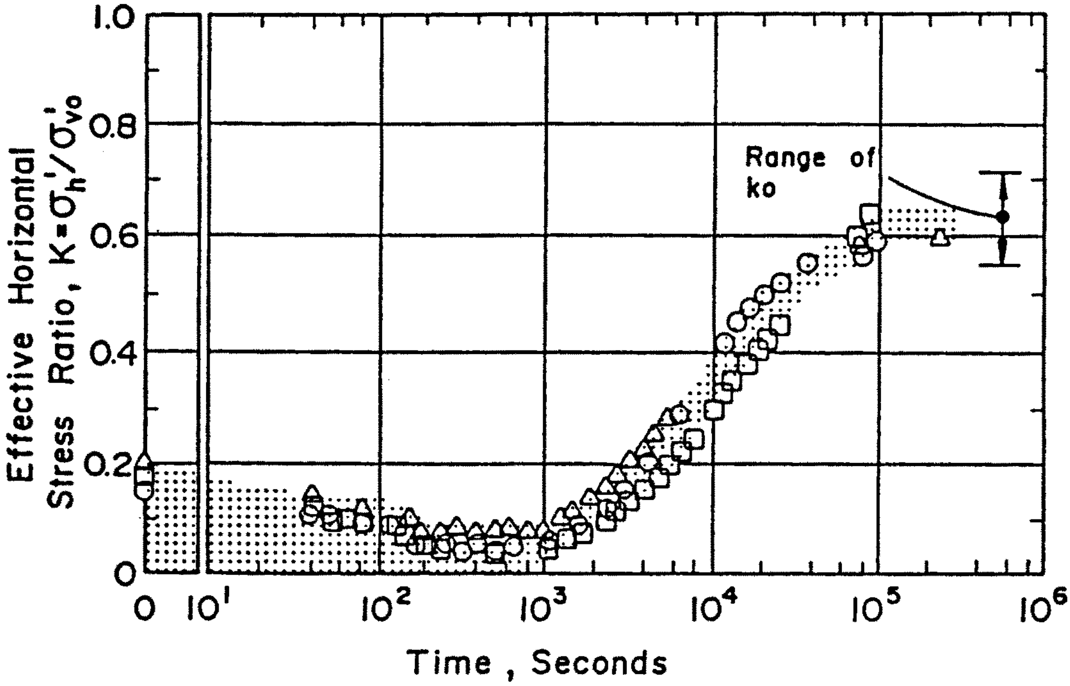

Figure 1. Normalized Measurements of Horizontal Effective Stress on Pile Shaft during Soil Consolidation following Installation in Lower Boston Blue Clay (After Azzou and Morrison, 1988)

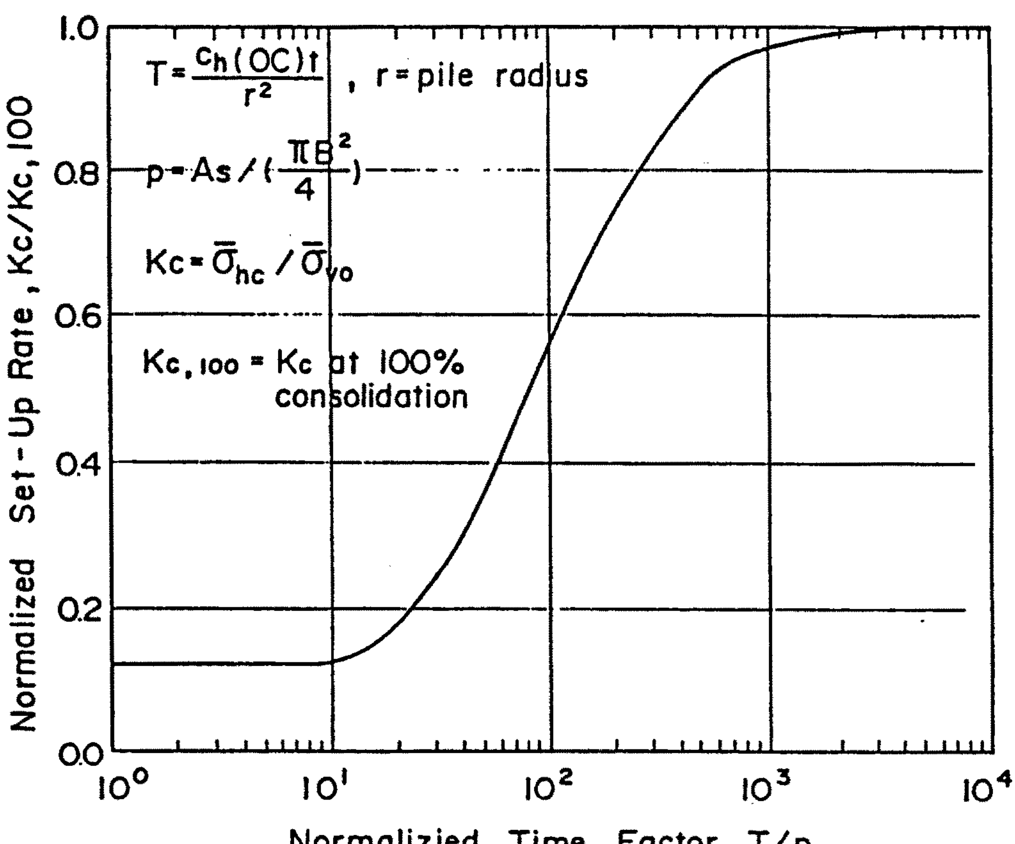

Figure 2. Normalized Set-up Curve

consolidated sample (3). Since no direct simple shear test was conducted, a value of 0.25 is used in this prediction based on judgment of available triaxial compression and extension tests.

It is also noted that a significant decrease in capacity (about 20%) should be expected if shearing to failure takes place after complete set-up of the pile because of a permanent decrease in effective stresses acting on the pile (1). However, the effect of partial consolidation is still not clearly understood. A simplified assumption has been made in this study that the reduction is proportional to the set-up rate, i.e., the reduction factor is equal to 0.2 times the ratio Kc/Kc,100 which can be obtained from Fig.2.

The prediction of the shaft resistance in clay was generally carried out according to the Kc method. Special considerations for each test pile are discussed in the following paragraphs:

(1) The Kc method can be properly applied to the driven steel pipe pile without further modifications.

(2) H pile is assumed to be a partially plugged pile with its contact perimeter equal to 2(bf+d) and cross section area of (bf x d), in which bf and d are the flange and depth of the HP 14x73 section, respectively. Based on Eq. 5, the parameter p can be calculated as 0.78. After this transformation, the Kc method can also be properly applied to the driven H pile.

(3) For the drilled slurry pier, the displaced soil (As in Eq.5) is relatively small and p value is expected to be close to 0. In addition, previous pile load test experiences indicate that the skin friction of bored pile is always smaller than that of the driven pile. Therefore, the Kc method is still used in predicting the skin friction of the slurry pier, but a 25% reduction has been applied.

(4) The prediction method of the cased pier is similar to that of the slurry pier. The difference is that a 40% reduction has been used instead of 25%. In addition, the pile diameter above 32 ft (9.75 m) is 24 in (0.61 m), whereas it is only 18 in (0.46 m) below 32 ft (9.75 m). It is assumed again that a pile length of 2.5 ft (0.76 m) below the casing depth will behave like a tapered pile shaft with unit skin friction 2 times that of a vertical shaft.

END BEARING IN CLAY

The ultimate end bearing of the pile in clay deposits is estimated by the LCPC method. It is recognized that the end bearing of each pile is also subjected to the effects of consolidation time and multiple shearing. In order to take this factor into account, an assumption was made that the influences of the set-up rate and multiple shearing effect on end bearing are treated similarly to those on shaft resistance in clay.

LOWER BOUND ESTIMATE WITH 90% CONFIDENCE

In order to obtain a lower bound estimate, some of the factors contributing to the variation of the predicted pile capacities have to be carefully evaluated. These important factors include the variation of soil thickness, the variation of soil properties, and the uncertainty of the prediction methods. Different considerations are applied to clay and sand so that major uncertainties can be incorporated in the lower bound estimate. Methods used to estimate the lower bound of the capacity are summarized below:

(1) The LCPC method is still used to estimate the skin friction of sand because the major uncertainty in sand is the variation of sand properties. The sand deposits are further divided into several layers. A lower bound estimate of q_c in each layer was determined as the q_c value which is smaller than 90% of the measurements in this layer. This lower bound q_c was substituted into LCPC method to yield the lower bound estimate of pile capacity with 90% confidence.

(2) In general, the lower clay deposits are relatively uniform. The major uncertainty is believed to be the prediction method. Previous experience of driven pile in Empire Clay indicates that the skin friction measurements obtained during cone penetration appear to provide reasonable estimates of the minimum skin friction of pile shaft (1). This conclusion is used to predict the 90% confidence capacity during first shearing of both the driven piles and bored piers. To take into consideration the multiple shearing effect, 10% and 30% reductions are applied to the second and third shearing, respectively. Since the end bearing in clay is almost negligible, no special consideration is applied to calculate the lower bound.

PREDICTIONS AND DISCUSSIONS

During pile load test, the ultimate skin friction and tip resistance will not be fully developed at the same time. Vijayvergiya's approach (8) is used in this study to take the mobilization effect into consideration, as shown in Figs. 3 and 4. The parameter z_c shown in Figs. 3 and 4 was determined in this study on the basis of previous working experience. It is suggested that the ultimate skin friction in sand and clay develops as the pile axial deformation reaches about 0.015B and 0.05B, respectively. The end bearing in clay fully develops at an axial deformation of about 0.15B.

Based on the rationale discussed above, predictions on the pile capacities are summarized in Table 4. The axial load distributions for 4 test piles are presented from Figs. 5 to 8. The lower bound

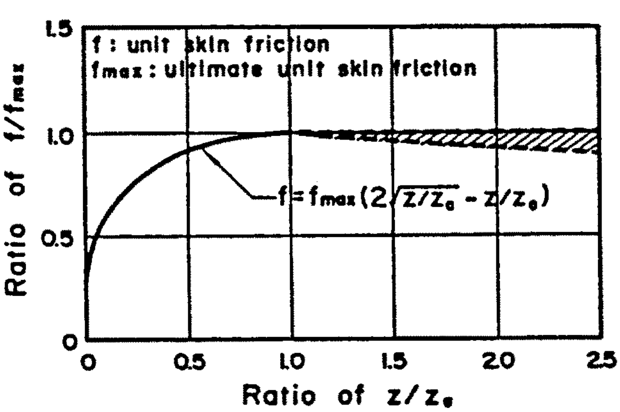

Figure 3. Normalized f-z Curve for Clay and Sand (After Vijayvergiya, 1977)

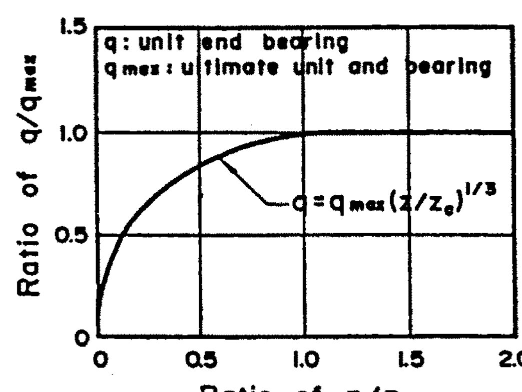

Figure 4. Normalized q-z Curve for Clay and Sand (After Vijayvergiya, 1977)

Table 4 Predicted Ultimate Capacities of Test Piles

Type of pile	Pile Capacity*, tons+		
	2 weeks	1 month	1 year
Steel pipe pile	70.2+9.5+1.8 =81.5	74.1+10.2+1.4 =85.7	78.0+18.2+2.3 =98.5
H pile	78.0+11.0+1.6 =90.6	78.0+11.3+1.3 =90.6	78.0+17.6+2.1 =97.7
Slurry pier	72.1+12.9+1.4 =86.4	72.1+10.3+1.1 =83.5	72.1+14.6+1.8 =88.5
Cased pier	76.9+21.1+1.4 =99.4	76.9+16.9+1.1 =94.9	76.9+18.0+1.8 =96.7

* friction in sand + friction in clay + end bearing in clay = ultimate capacity
+ 1 ton = 8.897 kN

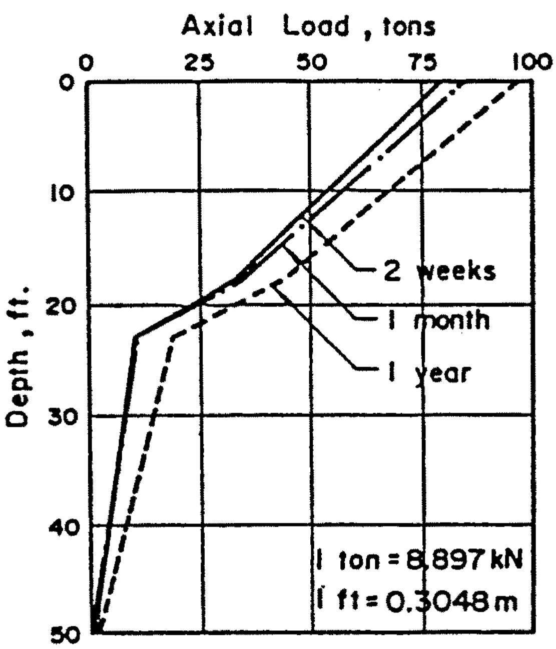

Figure 5. Axial Load Distribution at Predicted Failure Load (Steel Pipe Pile)

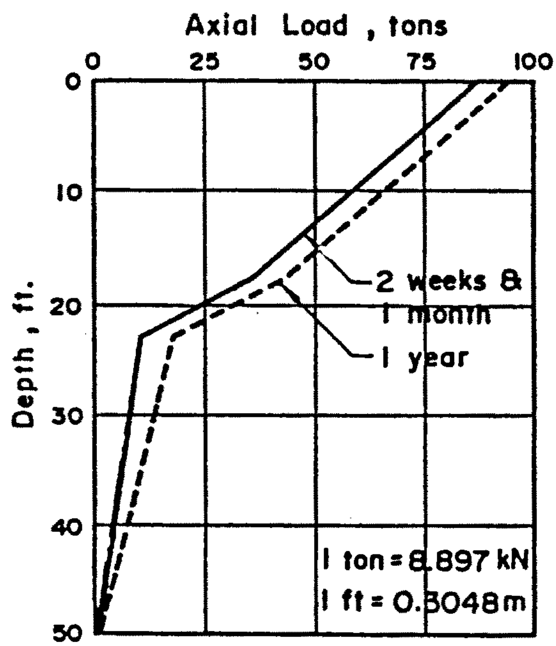

Figure 6. Axial Load Distribution at Predicted Failure Load (H Pile)

estimates are provided in Table 5, and also from Figs. 9 to 12. Some of the features of these basic predictions are discussed below:

(1) Since the end bearing in clay is small, the residual stress on the pile will also be insignificant. Therefore, this effect was neglected in prediction.

(2) The predictions indicate that sand deposits take most of the load.

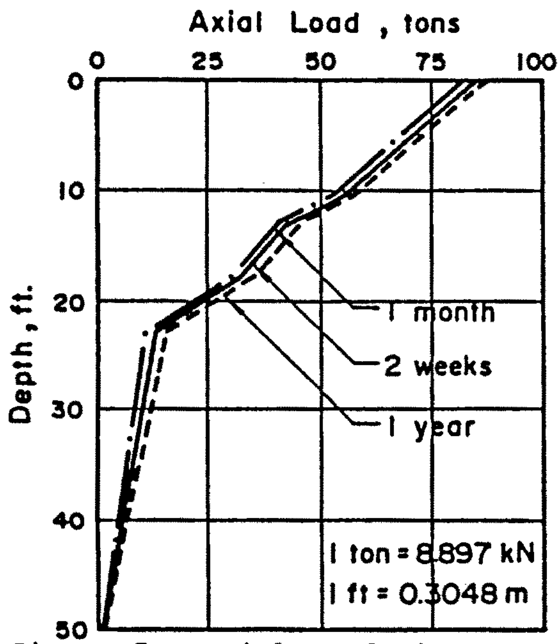

Figure 7. Axial Load Distribution at Predicted Failure Load (Slurry Pier)

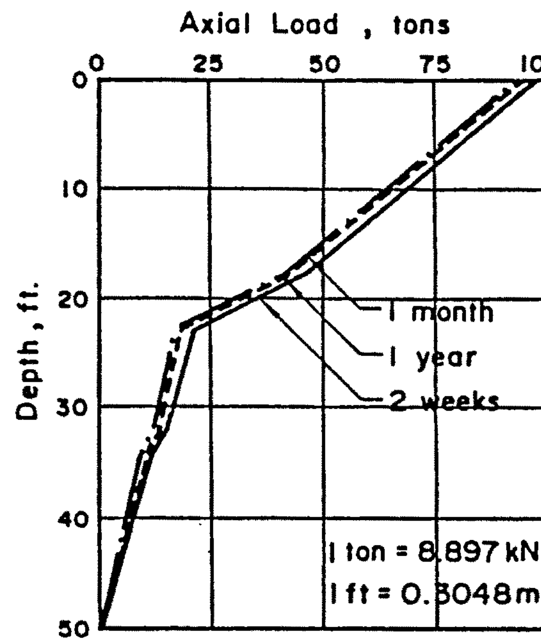

Figure 8. Axial Load Distributionat Predicted Failure Load (Cased Pier)

Table 5 Lower Bound Estimate Capacities of Test Piles

Type of pile	Pile Capacity*, tons+		
	2 weeks	1 month	1 year
Steel pipe pile	50.2+8.8+1.8 =60.8	53.0+7.9+1.4 =62.3	55.8+6.1+2.3 =64.2
H pile	55.88.8+1.6 =66.2	55.8+7.9+1.3 =65.0	55.8+6.1+2.1 =64.0
Slurry pier	51.6+9.1+1.4 =62.1	51.6+8.2+1.1 =60.9	51.6+6.4+1.7 =59.7
Cased pier	55.0+11.4+1.5 =67.9	55.0+10.3+1.2 =66.5	55.0+8.0+1.8 =64.8

* friction in sand + friction in clay + end bearing in clay = ultimate capacity

\+ 1 ton = 8.897 kN

(3) For all 4 test piles, the predicted pile capacities at 2 weeks, 1 month and 1 year after installation are not much different.

(4) At 1 year after pile installation, the predicted ultimate pile capacities for these 4 piles are not significantly different.

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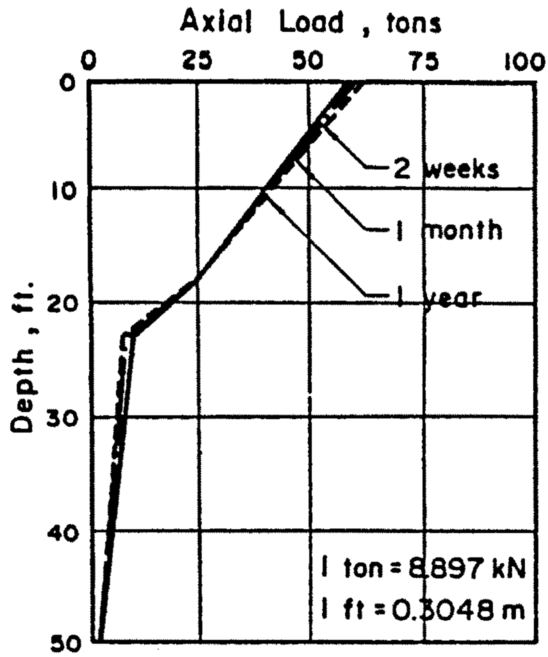

Figure 9. Axial Load Distribution at Lower Bound Capacity (Steel Pipe Pile)

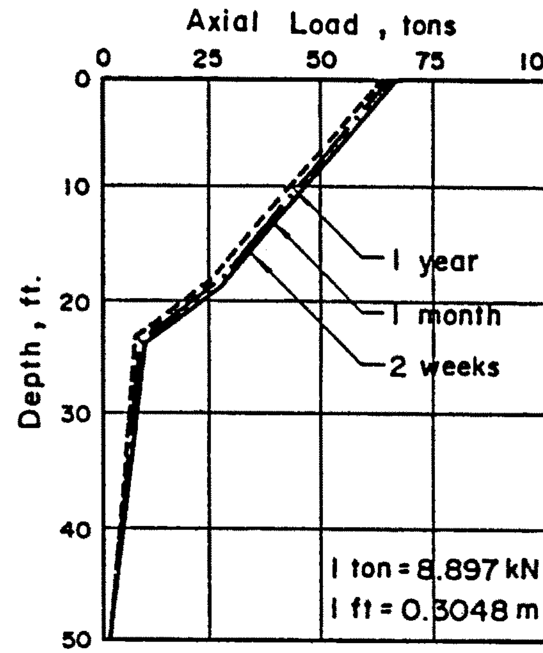

Figure 10. Axial Load Distribution at Lower Bound Capacity (H Pile)

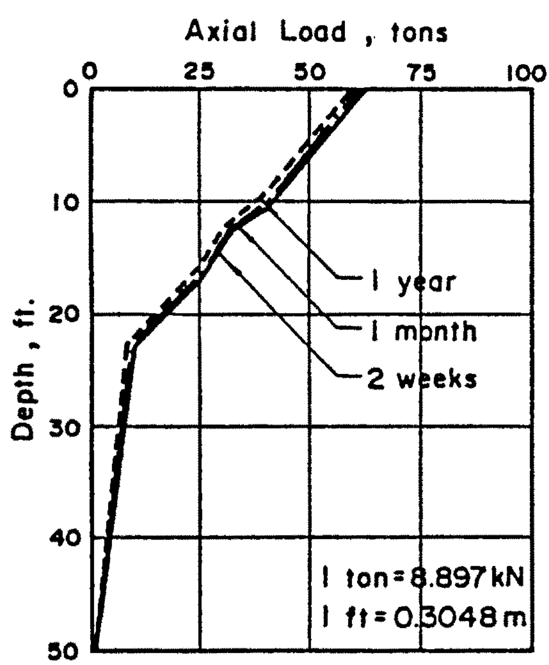

Figure 11. Axial Load Distribution at Lower Bound Capacity (Slurry Pier)

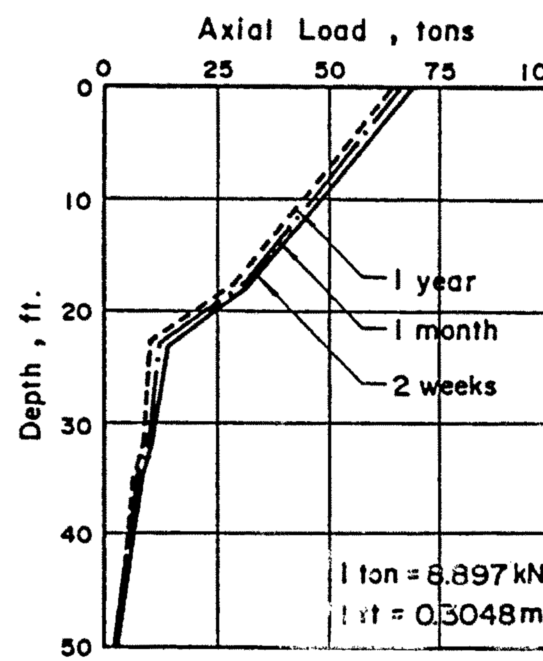

Figure 12. Axial Load Distribution at Lower Bound Capacity (Cased Pier)

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