Instrumented Load Test on a Bent Pile

P. Dunlop
Stone & Webster Engineering Corp., New York, New York

R. E. Sandiford
The Port Authority of New York & Jersey, New York, New York

D. R. Erali
Gilbert/Commonwealth Inc. Reading, Pennsylvania

Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, Paper No. 1.08

Instrumented Load Test on a Bent Pile

P. Dunlop
Senior Consulting Engineering, Stone & Webster Engineering Corp., New York, New York

D. R. Erali
Manager, Civil/Geotechnical Engineering. Gilbert/Commonwealth Inc. Reading, Pennsylvania

R. E. Sandiford
Senior Engineer, The Port Authority of New York & Jersey, New York, New York

SYNOPSIS The load carrying capacity of a bent shell pile in soft silts and clays was determined from an instrumented test. Lateral pile displacements along the pile were measured during loading and unloading using an inclinometer traveling in a plastic casing.

Pile capacity was estimated by Johnson's (1962) method prior to the load test and by the STRUDL structural engineering program after performing the load test. Both methods adequately predicted the pile performance. STRUDL, however, accommodated more realistic soil parameter variation and boundary conditions necessary for an integral soil-pile-structure interaction analysis.

INTRODUCTION

Driving thin shell piles through soft soils often results in a bent pile. The load carrying capacity of such piles is always of concern. Several procedures for calculating the load capacity of bent piles have been presented in the literature; Parsons et al. (1956), Johnson (1962), and Broms (1963).

This paper presents the analysis of a bent pile and the results of a specially instrumented load test. The pile was a TPT (tapered pile tip) composite pile consisting of a prefabricated, reinforced concrete tip of truncated cone shape attached to a 16 in. (0.4m) diameter corrugated steel shell. The shell and tip assembly was driven by means of a Vulcan-010 single acting pile hammer with an expandable steel mandrel.

The test pile was one of 1,300 piles driven for support of six fuel oil storage tanks. Each pile was designed as an end bearing unit of 150 ton (1335kN) design capacity, and was driven to an ultimate capacity of 300 tons (2670kN). Prior load tests had been performed to verify the specified driving criteria. A visual inspection of the unconcreted pile was performed after each production pile had been driven. A number of piles developed bends (sweeps) during driving. The degree of sweep encountered normally would not have been a cause for alarm; however, two factors were present that warranted the time and expense of an additional load test.

The first concern was that driving the TPT piles produced an annular space of approximately 6 1/2 in. (16.5cm) along the length of the pile. This was due to the difference in diameters between the enlarged pile tip and the corrugated steel shell. This annular space was backfilled with sand, although no special placement or compaction procedures were employed. It was believed that the sand backfill was limited to the top 20 to 25 ft (6.1m - 7.6m) of the pile, with the underlying soft soils filling the annular space below that depth. Since the degree of sweep became more severe with depth, there was a possibility that inadequate backfilling of the void was occuring where it was most needed.

The second concern was due to the extremely poor quality of the insitu soils in the vicinity of the swept piles. These soils consisted of flyash fill from the ground surface to a depth of approximately 10 feet (3m). Underlying the flyash was 32 ft (9.7m) of very soft silts and silty clays. These soils fell along the Casagrande A Line and the softer soils had liquid limits of 130 to 165 percent with natural water contents of 110 to 130 percent. Oedometer tests showed this stratum to be normally consolidated, and unconsolidated undrained (U-U) triaxial test data indicated shear strengths from 190 to 460 psf ($9.1kN/m^2$ - $22.1kN/m^2$). These soft soils were underlain by very dense, sands which served as the bearing stratum for the piles.

Figure 1 shows the simplified stratigraphic section and the initial bent shape of the test pile. The initial pile sweep and values for the radius of curvature R, along the pile were calculated from data obtained using a skid-mounted slope indicator sliding inside the TPT corrugated steel shell.

INSTRUMENTATION AND TESTING PROCEDURE

Prior to concrete placement in the test pile, a plastic casing was lowered to the base of the TPT socket. The pile was filled with concrete from the top by discharging through a funnel.

The load test was conducted in general accordance with ASTM "Standard Method of Testing

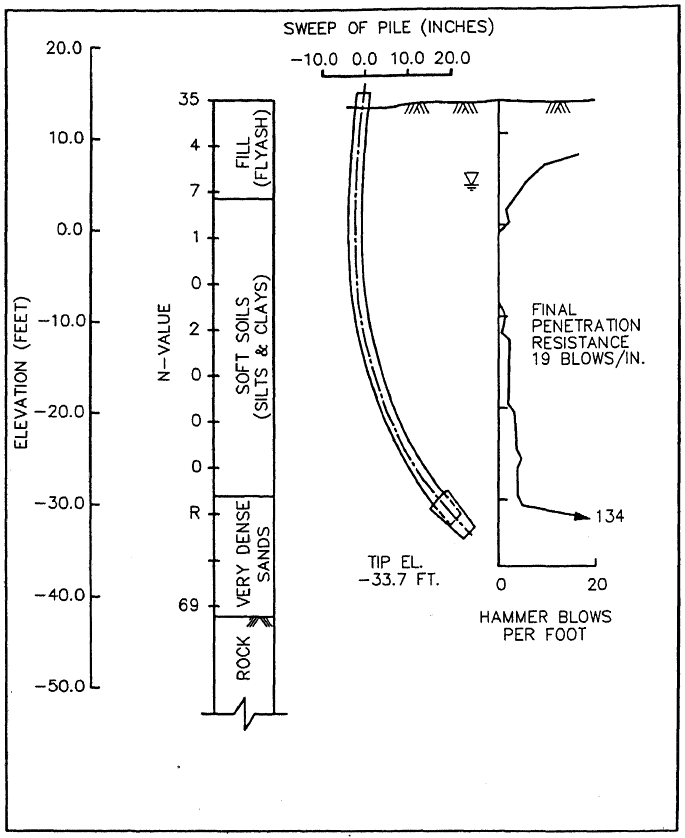

FIG.1 - TEST PILE AND STRATIGRAPHIC DATA

The loads were applied by means of a hand operated, 800 ton (7120kN) capacity hydraulic jack acting against a weighted platform. An electronic load cell, installed between the jack and a sperical bearing plate, was used to measure the load on the pile. Vertical pile butt movements were measured by four independently supported dial gages sensitive to 0.001 in. (0.0025cm). Lateral pile displacements along the pile were measured during loading and unloading using an inclinometer traveling in a plastic casing.

The test pile was loaded in 37.5 ton (334kN) increments to 300 tons (2670kN) (twice the design load). Each load increment, was to be maintained until the rate of butt settlement was less than 0.01 in. (0.025cm) per hour or until two hours had elapsed, whichever occurred first.

PRELIMINARY ANALYSES

Before load testing, potential pile behavior was investigated using a method proposed by Johnson (1962). This method idealized the bent pile as a laterally loaded beam on an elastic foundation. The lateral loading was calculated from

$$w = \frac{P}{R} \tag{1}$$

where
w = lateral load per unit of length, lb/ft
P = compressive pile load, lb
R = radius of curvature of the pile, ft

The test pile flexural rigidity, EI, was 12.4 x 10^9 lb-in^2 (35 x 10^6N-m^2) based upon a 5750 psi (3.97kN/cm^2), 10 day concrete strength. Little contribution to flexural rigidity was expected from the thin corrugated steel shell.

The Johnson analysis required the use of the coefficient of subgrade reaction, k_h. This parameter was estimated from the results of previous studies on soft soils in the area (Peck and Davisson, 1961) and from site specific oedometer and triaxial test data.

Peck and Davisson (1961) investigated the constant of horizontal subgrade reaction, n_h, in soft silts in the New York City area. They concluded that n_h ranged between 0.4 and 1.1 pci (0.01N/m^3 -0.03N/m^3). Thus, k_h was calculated from

$$k_h = n_h \cdot \frac{Z}{B} \tag{2}$$

where

Z = depth below surface, in.

B = width of pile, in.

This resulted in an estimated range of k_h for the soft soils of 10 to 30 pci (0.027N/m^3 - 0.081N/m^3) when corrected for water table elevation and stratigraphy.

Terzaghi (1955) suggested calculating k_h from elasticity and proposed

$$k_h = \frac{E_s}{1.35B} \tag{3}$$

where

E_s = soil modulus , psf, and

B = the pile diameter, ft

A similar relationship was proposed by Vesic (1961).

The value of effective pile width, B, could conceivably be larger than the shell width if the annular space around the corrugated shell (caused by driving the enlarged concrete tip) were completely filled with sand backfill. Observations in the field indicated that the soft soils were probably filling this void, particularly at depth where the test pile had the smallest radius of curvature and therefore B was taken as the corrugated shell diameter.

Values of soil modulus, E_s, were calculated from several laboratory oedometer and triaxial tests. E_s from oedometer test results ranged from 6000 to 39,000 psf (288kN/m^2 - 1872kN/m^2). An average value of 22,600 psf (1084kN/m^2) was considered representative for the softer soils in the range of overburden stresses. The oedometer and triaxial tests were performed on specimens trimmed with their axes vertical. The soil modulus should reflect horizontal pile loading, therefore stress levels were reduced to

Koσ_v', where Ko is the coefficient of lateral earth pressure at rest, estimated from 1-sinϕ', and σ_v' is the effective overburden stress. Initial E_s values were calculated at two percent strain.

Values of E_s were also calculated from the results of vertically trimmed unconsolidated undrained (U-U) triaxial tests. These values were more indicative of undrained conditions as would be the case for rapid pile loading. Values of E_s calculated by this procedure ranged from 7000 psf to 50,000 psf (336kN/m^2 - 2400kN/m2), with the average near 20,000 psf (960kN/m^2). The predicted k_h values from these data range from 2 pci to 16 pci (0.05N/m^3 - 0.43N/m^3), with an average value near 8 pci (0.22N/m^3).

Davisson and Robinson (1965) proposed calculating the horizontal subgrade modulus k* from

$$k^* = 67\ Su \qquad (4)$$

where

k* = horizontal subgrade modulus = k_hB, lbs/in^2

Su = undrained shear strength, lbs/in^2.

Assuming an average shear strength of 250 psf (12kN/m^2), the value of k_h using this approach is about 7 pci (0.19N/m^3).

The Johnson analyses were therefore run with a range of k_h values from 4 pci to 25 pci (0.11N/m^3 - 0.68N/m^3) to investigate the possible range of pile behavior.

RESULTS OF PRELIMINARY ANALYSIS

The figures referenced in the following sections show the results of the Johnson analyses which were performed before the pile load test, the load test measured data, and the results of the STRUDL analyses which were performed after completion of the pile load test.

The end of increment load-settlement-time relationship is presented in Fig. 2. A plot of lateral displacements versus depth was generated from the inclinometer data and is shown in Fig. 3. Figure 4 shows the maximum lateral pile displacement versus pile load.

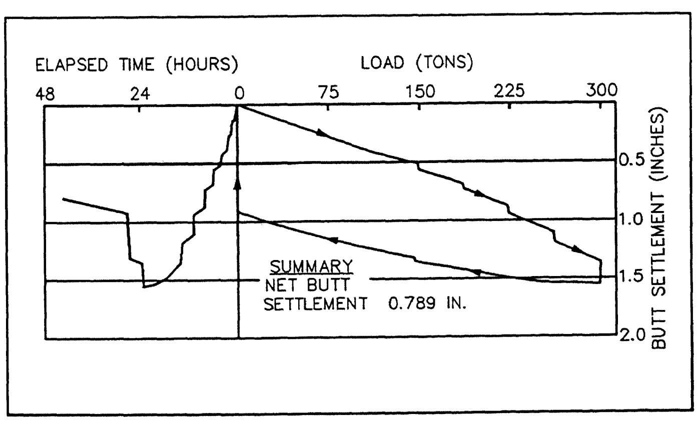

FIG.2 – LOAD–SETTLEMENT–TIME DATA

Figure 5 shows predicted lateral pile displacements calculated using Johnson's Method for 150 and 300 ton (1335kN - 2670kN) pile loads

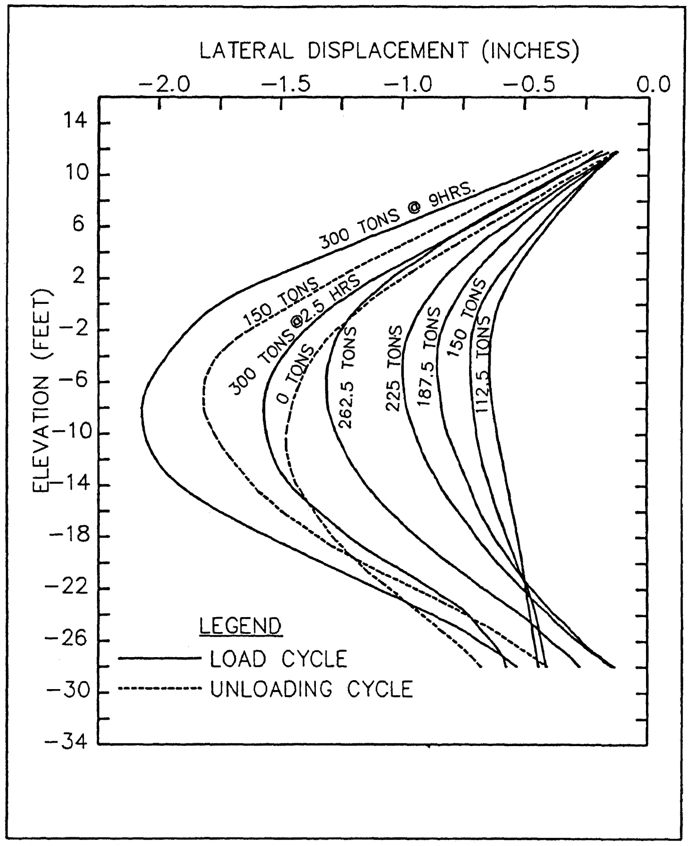

FIG.3 – OBSERVED LATERAL PILE DISPLACEMENT

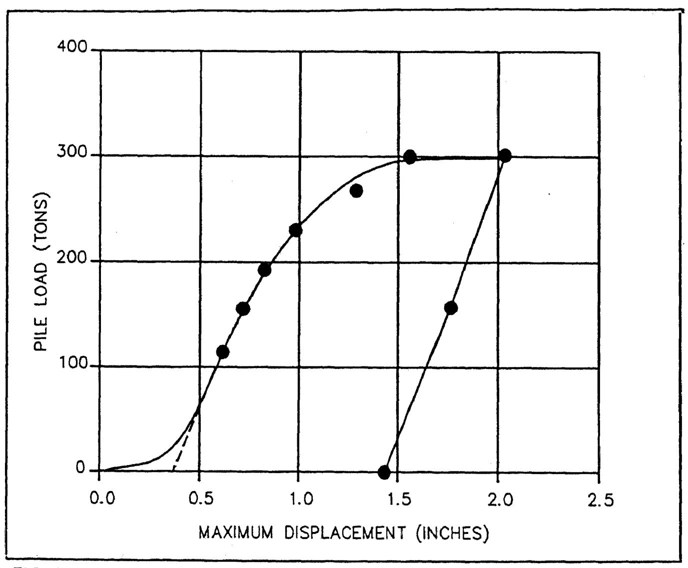

FIG.4 – PILE LOAD VS MAXIMUM LATERAL DISPLACEMENT

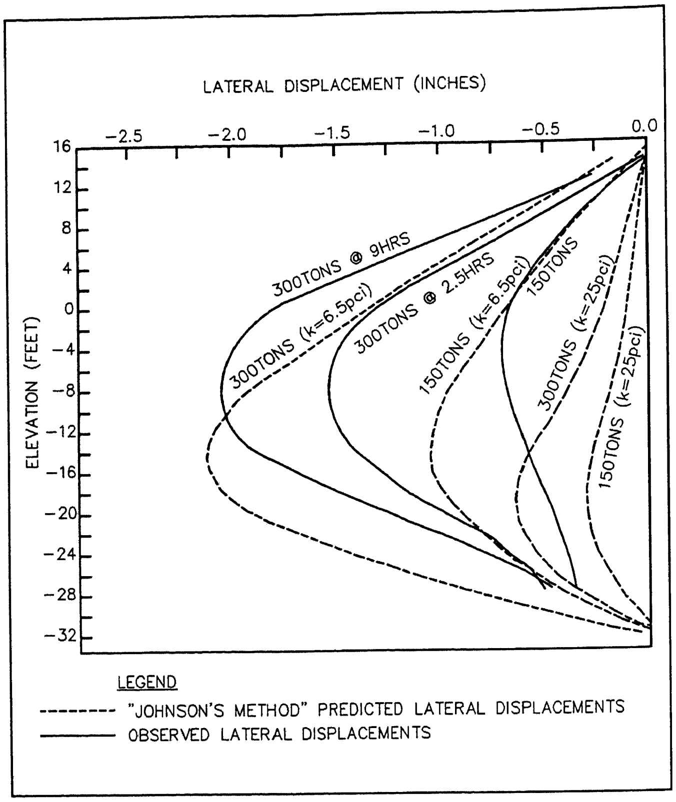

FIG.5 – PREDICTED – JOHNSON VS OBSERVED LATERAL PILE DISPLACEMENT

with k_h values of 6.5 and 25 pci ($0.18N/m^3$ - $0.68N/m^3$). The displacements calculated with k_h = 25 were significantly lower than those subsequently measured during the pile load test. Predicted displacements at 150 tons (1335kN) with k_h = 6.5 were higher than those measured and the predicted displacements at 300 tons (2670kN) with k_h = 6.5 were close to the measured 9 hr. displacements. This is consistent with the fact that the effective k_h will decrease as strain increases.

The Johnson analysis was also used to predict soil stresses along the pile and bending moments in the pile. Figure 6 shows a plot of predicted soil stresses along the pile for the 150 and 300 ton (1335kN - 2670kN) loads. Values shown were calculated with k_h = 25 pci ($0.68N/m^3$); somewhat lower soil stress values were obtained with lower k_h values but the difference was not significant. The maximum calculated soil stress at the 150 ton (1335kN) load is about one half of the assumed ultimate lateral soil resistance of 9c (Broms, 1965), in which c is the undrained shear strength. Slightly lower values of ultimate lateral soil resistance (7.5c to 8c) were obtained by the procedure recommended by Davisson and Prakash (1963). At the 300 ton load (2670kN) the soil stress is greater than 9c implying that the soil would be overstressed and failing.

The predicted pile bending moments were also checked. Figure 7 shows a plot of calculated load eccentricities for various k_h values and also the ultimate ACI short column design values. Results labelled C, D and E were calculated with K_h = 25, 6.5, and 4 pci ($0.68N/m^3$, $0.18N/m^3$, and $0.11N/m^3$), respectively. The predicted load eccentricity was constant for all pile loads when k_h was held constant.

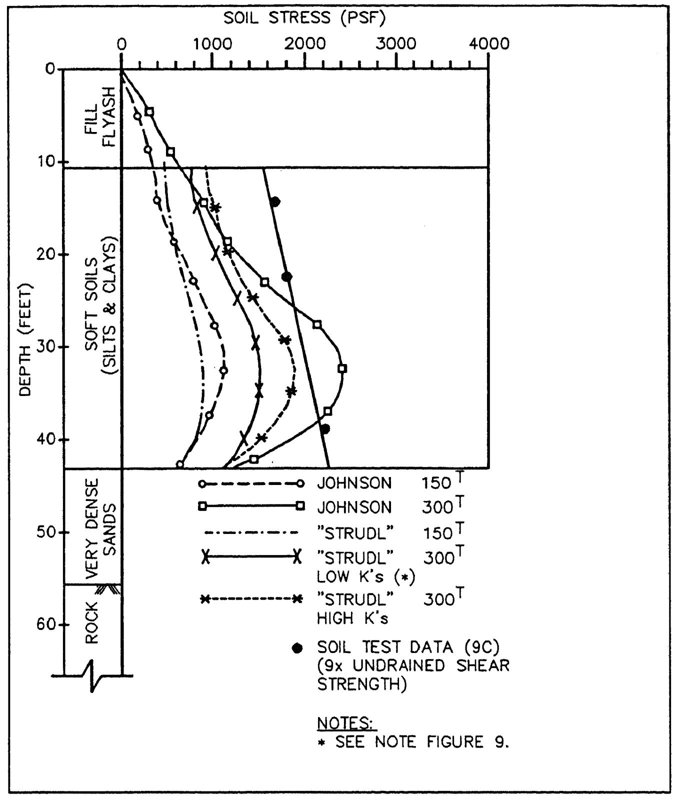

FIG.6 – PREDICTED SOIL STRESSES ALONG PILE

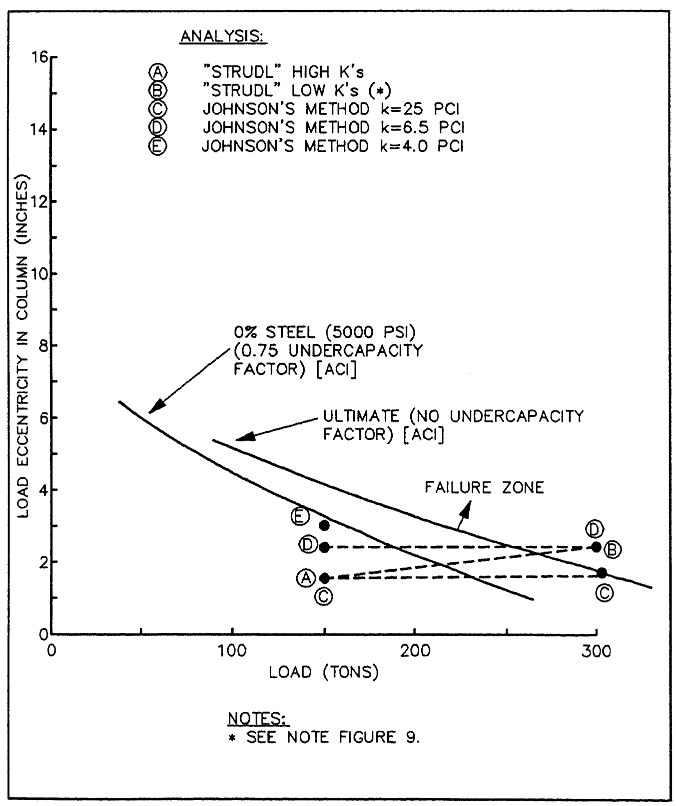

FIG.7 – SHORT COLUMN ANALYSIS OF PILE

Therefore, the calculated load eccentricity for k_h = 25 pci (0.68N/m^3) is 1.5 inches (3.8 cm) at both the 150 and 300 ton loads (1335kN and 2700kN). These results show that calculated eccentricities are less than the short column ultimate value at 150 tons (1335kN) but more than this value at 300 tons (2670kN).

Although not discussed herein Broms (1963) procedure for analyzing bent piles also adequately predicted the test pile performance but like Johnson's (1962) method suffers from lack of versatility.

The analyses therefore, predicted a stable pile at the design load but failure at the test load. Clearly care was called for during performance of the pile load test.

LOAD TEST RESULTS

The measured lateral displacements versus depth are presented in Figure 3 and show a reasonably consistent trend. The maximum lateral deflection versus load has been plotted in Figure 4. An abnormally large initial displacement appears to have occurred at low loads and may be related to incomplete backfilling of the annular space around the pile or impact on the soft soils of driving the large TPT pile tip. Instrument accuracy, pile butt movement and reproducibility of readings may also have been contributing factors. It is clear from Figure 4 that a significant disproportionate lateral displacement was occurring above about 225 tons (2000kN) and that the pile was continuing to move laterally at the 300 ton (2670kN) load and was probably failing.

The unload curve was essentially linear. No lateral displacements were observed during the 150 ton (1335kN) load hold. In fact, as shown in Figure 2, the pile butt was continuing to rebound showing that the pile was not yielding as it had been at the 300 ton (2670kN) load.

The Johnson analysis therefore had provided a good prediction of the test pile behavior. However, Johnson's method did not allow for general boundary conditions (applied shears, bending moments, or displacements) nor did it allow for variations in k_h due to stratigraphy or lateral displacements. For these reasons the STRUDL program was used to analyze the test pile and was subsequently used for analyses of other production piles with significant bends.

STRUDL ANALYSES

Parameters used in the STRUDL analyses are shown in Figure 8. Spring constants were calculated from

$$K = k_h \, LB \qquad (5)$$

where
k_h = coefficient of subgrade reaction, pci
L = distance between support springs, inches
and
B = pile diameter, inches

The value of k_h is not a unique physical property. In addition to varying as the inverse of pile diameter, the value of k_h varies with soil modulus, which is a function of soil type and stress and strain history.

The 10 feet (3m) of flyash fill overlying the soft silts and clays was assumed to behave as a silt and a value of n_h was estimated to be about 5 pci (0.14N/m^3).

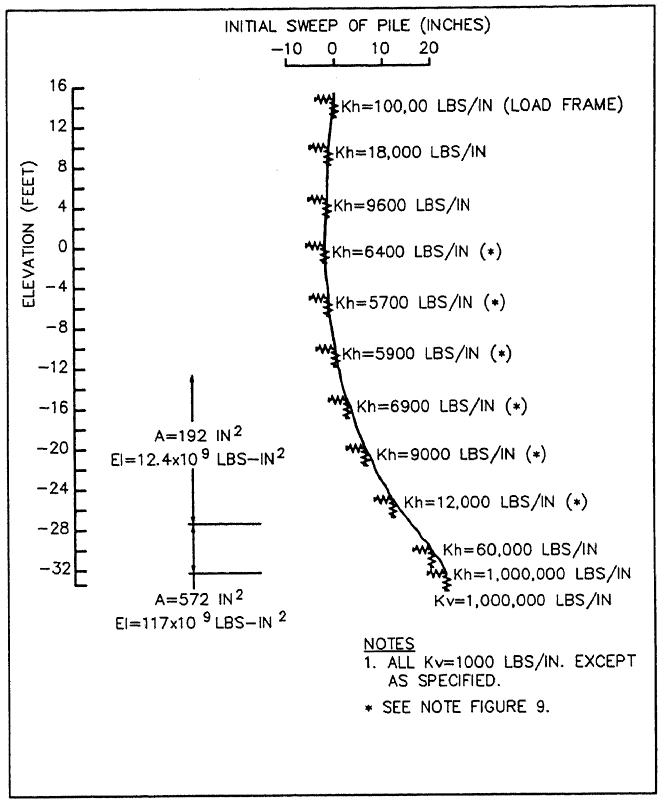

FIG.8 - STRUDL MODEL

The value of k_h decreases as the strain level or displacement increases. For soil near the ground surface the limiting stress is reached when a soil wedge is formed and forced upward in front of the pile. Beyond a certain depth, the limiting stress is reached when a flow type failure mode results (Broms, 1965; Davisson et al, 1963; and Reese et al, 1974). Analyses showed that the critical failure mode for the test pile was a flow failure in the soft soil.

Stress-displacement (p-y) curves for the soft soils were calculated assuming an ultimate soil stress value of 9c and a p-y variation similar to, but lower than, that suggested by Audibert and Nyman, (1977). The ultimate lateral displacement was assumed at be 1.5 inches (3.8cm). Initial k_h values for the STRUDL ANALYSES were estimated from the variation in measured lateral displacements (Fig. 3) and the p-y curves. (An iterative approach could have been performed). The value of the load frame spring was selected to represent a significant restraint. The next spring (Fig. 8) reflected estimated flyash behavior. The third spring was estimated for clay and flyash response because this spring was at the interface of these two materials. Below this, spring constants represented soft clay and silt. Soil springs near the base of the pile were estimated from bearing capacity analyses and from earlier measured pile tip load-displacement data obtained from piles tested earlier in the project.

The STRUDL predicted lateral pile displacements are shown in Figure 9 and are close to the measured values. Displacements of the test pile at the 300 ton (2670kN) test load were continuing as shown in Figures 3 and 4. An analysis at 300 tons (2670kN) with reduced k_h values was run to represent this case and is also shown on Figure 9. The displacements were larger than the 300 ton (2670kN) 9 hr. readings, however, as mentioned the test pile was continuing to deflect laterally and would have reached and exceeded these calculated values had the test load been maintained for a longer period of time.

Soil stresses were calculated and are shown in Figure 6. The soil stresses at 150 tons (1335kN) were about one-half the predicted ultimate values whereas the stresses at 300 tons (2670kN) approached the ultimate value of 9c. Calculated stresses for the analysis with reduced k_h values were quite a bit below the ultimate value. This was a fictitious result and does not imply stability as the induced bending moments in the pile were larger with a lower k_h value as shown in Figure 7. The calculated eccentricity for the best estimate of k_h variation (analysis A) was about 1.5 inches

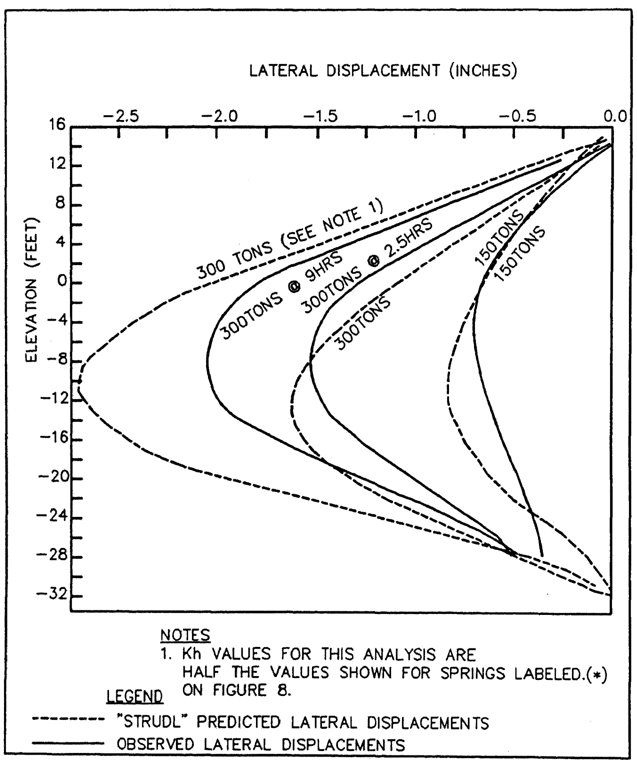

FIG.9 – PREDICTED – STRUDL VS OBSERVED LATERAL PILE DISPLACEMENT

(3.8cm) (stable) whereas it was 2.3 inches (5.8cm) (unstable) with the softer springs (analysis B).

The conclusion from the STRUDL analyses was that the pile was stable at 150 tons (1335kN) but that it was failing at 300 tons (2670kN). The failure probably started between 225 and 250 tons (2000kN - 2225kN).

CONCLUSION

The load test showed the bent TPT pile to be capable of supporting the 150 ton (1335kN) design load but not capable of sustaining the 300 ton (2670kN) test load.

The pile load test showed that both the Johnson (1962) procedure and the STRUDL analyses predicted the test pile performance when representative soil parameters were used. The STRUDL analysis was relatively simple yet accommodated complex stratigraphy and boundary conditions.

These analyses also showed that the conclusions of pile behavior were not very sensitive to the value of the coefficient of subgrade reaction. This is demonstrated by the fact that the calculated maximum pile capacities were similar for a wide range of k_h, even though the corresponding values of predicted lateral displacements and predicted soil stresses were substantially different.

REFERENCES

American Concrete Institute (1970), Ultimate Strength Design Handbook, Vol. 2, Column, Special Publication No. 17A

Audibert, Jean M.E., and Nyman, Kenneth J. (1977), Proceedings, ASCE, Vol. 103, No. GT10, Oct., p. 1119

Broms, Bengt B. (1963), Proceedings ASCE, Vol. 89, No. SM5, Sept., p. 73.

Broms, Bengt B. (1965), Proceedings, ASCE, Vol. 91, No. SM3, May, pp. 79

Davisson, M.T., and Prakash, S. (1963), Proceedings of the Highway Research Record, No. 39, p. 25

Davisson, M.T., and Robinson, K.E. (1965), Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, p. 243

Johnson, Sidney M. (1962), Proceedings ASCE, Vol. 88, No. SM6, Dec., P. 65.

Parsons, James D., and Wilson, Stanley, D. (1956), Transactions, ASCE, Vol. 121, p. 695.

Peck, R.B., and Davisson, M.T. (1961), Transactions, ASCE, Vol. 128, Part IV, p. 413.

Reese, Lyman C., et al. (1974), Proceedings of the 6th Offshore Technology Conference, Vol. 2, p. 473

STRUDL - ICES STRUDL II (1968), The Structural Design Language, Vol. 1, Frame Analysis, Department of Civil Engineering, Massachusetts Institute of Technology, Nov.

Terzaghi, Karl (1955), Geotechnique, Vol. 5, No. 4, p. 297

Vesic, A.S. (1961), Proceedings, ASCE, Vol. 87 No. EM2, April, p. 35