Laterally Loaded Piles in Layered Soils

Pieux sous Charge Latérale dans les Sols Stratifiés

L.C. REESE T. U. Taylor Professor of Civil Engineering, The University of Texas at Austin, Austin, Texas
J.D. ALLEN Design Civil Engineer, ARCO Oil and Gas Company, Dallas, Texas
J.Q. HARGROVE Staff Engineer, Woodward Clyde Consultants, Baton Rouge, Louisiana

SYNOPSIS A series of experiments were performed to investigate the behavior of piles under lateral loading in layered soils. Tests were conducted with a 25 mm diameter pile in the laboratory and with a 152 mm diameter pile in the field. Experimental results were compared to results from analyses, and fair to good agreement was obtained with the assumption that the behavior of the individual layers was the same as if all of the soil were uniform in characteristics.

INTRODUCTION

A method of analysis in widespread use for predicting the behavior of laterally loaded piles involves the solution of the equation of beam bending.

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} - p = 0 \qquad (1)$$

where EI = flexural stiffness of the pile, y = lateral deflection, x = distance along the pile, P_x = axial load, and p = soil resistance. The equation can be expressed in finite resistance form and solved by computer. The soil resistance as a function of deflection is expressed in the form of p-y curves. Criteria for constructing p-y curves have been presented by Matlock (1970) for soft clay, Reese et al. (1975) for stiff clay below the water table, Welch and Reese (1975) for stiff clay above the water table, Reese et al. (1974) for sand, and Sullivan et al. (1979) for clay below the water table. These authors have presented recommendations for both static and cyclic loading. Gill and Demars (1970) have presented recommendations for p-y criteria for static loading.

Most of the p-y criteria are based on the results of lateral load tests in homogeneous soils, coupled with earth pressure theory. Results of analyses based on these criteria have been compared to additional field load test results by Stevens and Audibert (1979), Bhushan et al. (1979), Ismael and Kylm (1978), Lee and Gilbert (1979), and Meyer (1980). Suggestions for modifications have been made by Stevens and Audibert (1979), Lee and Gilbert (1979), Meyer (1980), and Sullivan (1977). These evaluations of the p-y criteria for homogeneous soils have brought some degree of confidence to the use of these methods.

Only a limited amount of research has been done on resistance to lateral movement of a pier or pile in layered soils (Naik and Peyrot, 1976; Davisson and Gill, 1963; Dordi, 1977; Khadilkar et al., 1973). No published experience is available to the engineer who must select p-y curves for the layered soil conditions. Research at this university is aimed at developing recommendations for p-y curves in layered soils.

LABORATORY EXPERIMENTS

Small-scale tests were carried out using a 25 mm diameter aluminum pile in a plastic-lined tank of soil. The tank was 1.07 m square in plan and about 0.8 m deep. Soil was placed in the tank to a depth of 0.76 m. The piles were embedded to within 124 mm of the bottom of the tank. Soft clay was placed in the lower portion of the tank and a layer of stiff clay was placed at the surface. Three dial gauges behind the pile were used to measure deflection and slope.

Details of the test pile are shown in Fig. 1. Electrical resistance strain gauges were attached to an inner tube on each side at 19 locations. The inner tube was then inserted into the outer tube. Spacers caused the tubes to act as one unit in bending. The pile was calibrated by applying known moments and recording the strain output at each station.

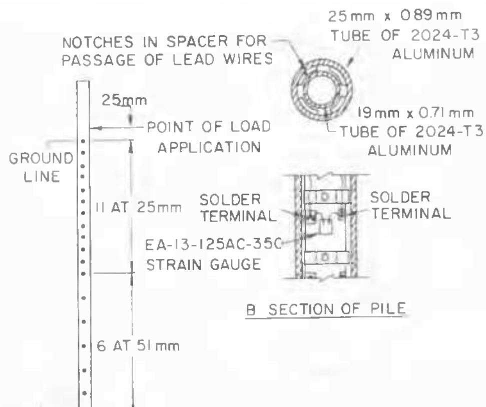

Fig. 1 Details of 25-mm diameter pile

The soil used in all of the tests had a liquid limit of about 56 and a plastic limit of about 22 and would be classified as a CH in the United System. The soft clay was placed by hand after being mixed with water to give an average moisture content of about 34%. The stiffening of the upper layer was achieved by mixing Type III Portland Cement with the soil at a moisture content of about 46% and running the mixture through a vacuum-extrusion machine. The extruded slugs were placed in the tank without creating a significantly irregular interface.

Vane tests were performed in all layers following each pile test and some additional unconfined and Q-type triaxial tests were performed on samples from the stiff layers after each test.

Five static-load tests were performed in a period of about six weeks. The soft soil was packed in the tank in a 3-day period to a depth of 0.76 m, the pile was jacked into the soil mass at the center of the tank, and Test 1 was performed using only the homogeneous soft clay in the tank. The pile was withdrawn, soil around where the pile had been was dug out and repacked so that the void left by the pile would be completely refilled, and enough soft clay was taken out to reduce its depth to 0.71 m. The pile was shoved into the mass again and a 51 mm thick layer of extruded cement-containing soil was placed on the surface. Test 2 was performed eight days after placement of the upper layer. Similar procedures were followed with the remaining three tests. Upper thicknesses of 76, 102, and 152 mm were placed for Tests 3, 4, and 5, respectively.

During the tests, strain recordings were made at thirty seconds, one minute, and five minutes after each increment of load was applied. Increments of load were generally added after six minutes had elapsed since the last increment.

Layer thicknesses and soil properties for the five tests are shown in Table I. The X-Coordinates are measured from the point of load application and ε_{50} is the strain at one-half the compressive strength in the undrained compression tests.

Table I Soil Properties and Layer Thicknesses

Pile Test No.	Layer	X-Coordinate at Top of Layer, mm	X-Coordinate at Bottom of Layer, mm	Shear Strength kN/m^2	ε_{50}
1	1	27	663	15.7	0.005
2	1	23	82	153	0.0031
	2	78	663	20	0.005
3	1	26	102	178	0.0031
	2	102	663	23.6	0.005
4	1	26	127	139	0.0031
	2	127	406	26.3	0.005
	3	406	663	23.7	0.005
5	1	26	179	183	0.0031
	2	179	210	29.6	0.005
	3	210	406	26.3	0.005
	4	406	663	23.7	0.005

Figure 2 is a plot of groundline deflection vs. load for the readings taken at the one-minute interval for all the tests. Also plotted in Fig. 2 are computed curves employing p-y curves. The computer program used for the analyses was COM623 (Sullivan, 1977), an adaptation of a program documented by Reese (1977). The p-y criteria for stiff clay above the water table (Welch and Reese, 1975) was used for the stiff material while that for soft clay (Matlock, 1970) was used for the soft material. The p-y curves were constructed for each layer as if the entire mass of soil had the properties of that layer.

Initial slopes of the measured and computed curves differ significantly in the case of Test 1, but compare very well in the tests on the layered soils.

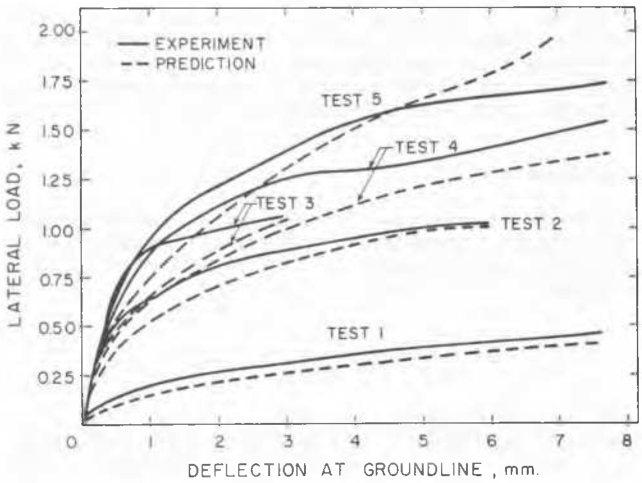

Fig. 2 Results for 25-mm diameter pile

FIELD EXPERIMENTS

At the site of the field tests, a loose, fine, uniform sand exists to a depth of about 55 ft. Beneath this alluvial deposit was limestone bedrock. The water table was normally at a depth of about 1.5 m; however, the lake was lowered at the time of the experiments and the water table was at about 3.6 m below the original ground surface.

To achieve a layered soil profile, an upper layer of clay was placed above the natural deposit of sand. The clay for several feet around the pile was placed in lifts of 5 to 8 cm and compacted with a hand-operated, vibratory compacter. The final thickness of the clay layer was 0.38 m.

The properties of the upper layer of clay were obtained from laboratory tests of samples that were obtained with thin-walled tubing and from field tests with a small-sized, vane-shear device. The properties of the sand were obtained from results of field tests using a dynamic penetrometer. The undrained shear of the clay was 96 kN/m^2 and its unit weight was 18 kN/m^3. The angle of internal friction of the sand was 34^{o} and its unit weight was 19.6 kN/m^3.

A steel tube, 6.43 m long and 152 mm in outside diameter with a 3 mm wall thickness, was used for the pile. The tube was split longitudinally and electrical-resistance strain gauges were installed along the inside wall of each half of the tubing. The gauges were carefully bonded; connected to a four-conductor, shielded cable; and then covered with a waterproofing compound. The two halves of the pipe were put together and welded. Care was taken to insure that the temperature increase due to welding kept low enough not to damage the instrumentation. Steel plates were attached to the top of the pile. The interior of the pile was pressured with nitrogen, a desiccant, and the leaks in the welds were found and sealed. The nitrogen pressure was maintained through much of the test period to insure that the strain gauges remained completely dry.

There were 32 strain-gauge stations with the first station at 1.55 m from the top of the pile. From that point, 22 of the stations were spaced 101 mm apart, 4 of the stations were 152 mm apart, 5 of the stations were at 305 mm apart, and the space to the final station was 457 mm. The last station was 152 mm above the bottom of the pile.

The upper, unloaded portion of the pile was used for the measurement of the pile-head rotation. Three levels of dial gauges were used for this purpose, as well as a slope indicator (see bracket in Fig. 3).

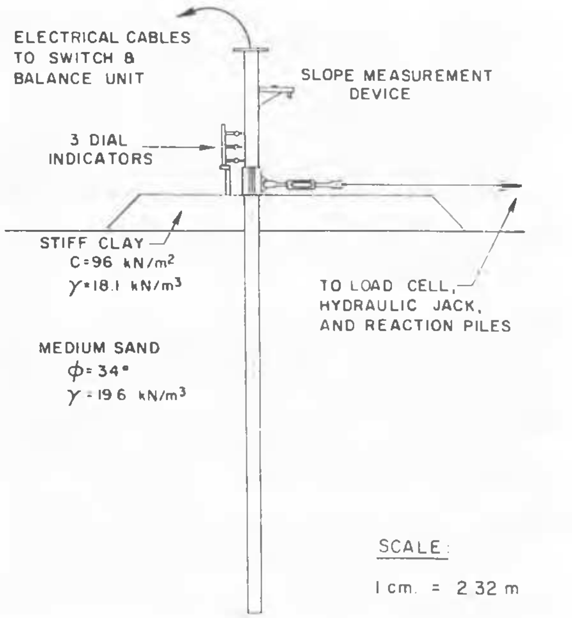

Fig. 3 Arrangement for testing 152-mm dia. pile

The pile was calibrated for bending moment by fixing it over supports in the laboratory and applying weights of known magnitude. Readings were taken with a strain-gauge indicator and a calibration constant was obtained for each station.

A special drop hammer was built for driving the pile. A rod, attached to the bottom of the hammer, served as a guide and plywood plates were used for a cushion block. A 100-mm diameter hole was augered by hand to a depth of 1.5 m to serve as a guide for the installation of the pile. Plumbness and alignment were checked as the pile was driven into position. After the pile was placed, the final testing arrangement, as shown in Fig. 3, was installed. A steel cable was attached to a previously-installed reaction system, consisting of two steel pipe piles. A hydraulic ram and a load cell were placed in series in the load line. A hand-operated, hydraulic pump was used to actuate the hydraulic ram.

The pile was loaded in increments of 1.1 kN up to 22.2 kN . The test concluded after 2 increments of 2.2 kN to 26.6 kN. Each load was applied to the pile 10 times and pile slope, deflection, and strain gauge readings were taken after the first, fifth, and tenth cycles. These readings were observed at approximately one-half minute and again at one and one-half minutes after the load was first applied. The strain gauge readings required about 35 seconds as the data acquistion unit balanced each of the bridge circuits and printed the output in microstrains.

Some results from the testing are shown in Figs 4 and 5. Figure 4 shows the pile-head deflection and the maximum bending moment for the first load cycle and Fig. 5 shows those values for the tenth load cycle. Predictions were again made by use of the computer with the assumption that the soil response was the same as if the soil conditions were uniform. That is, the p-y curves were derived for the clay as if the entire soil profile consisted of clay and the curves were derived in a similar manner for the sand.

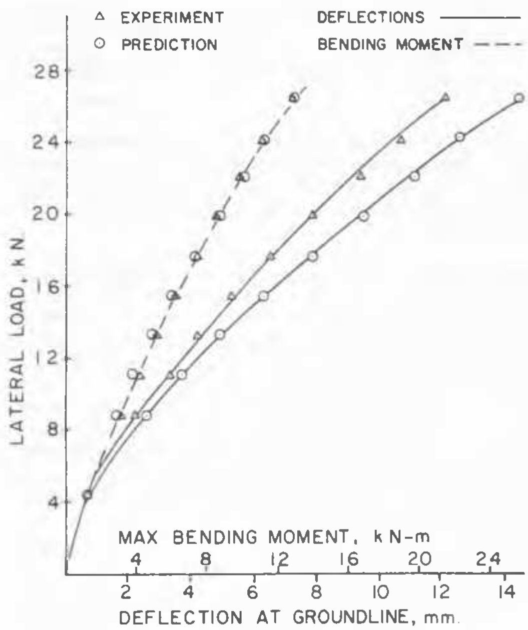

Fig. 4 Results for 152-mm dia. pile, first load cycle

As may be seen, excellent agreement was obtained between computed and experimental results for bending moment for both cases. The computed deflections are somewhat greater than those obtained in the experiment.

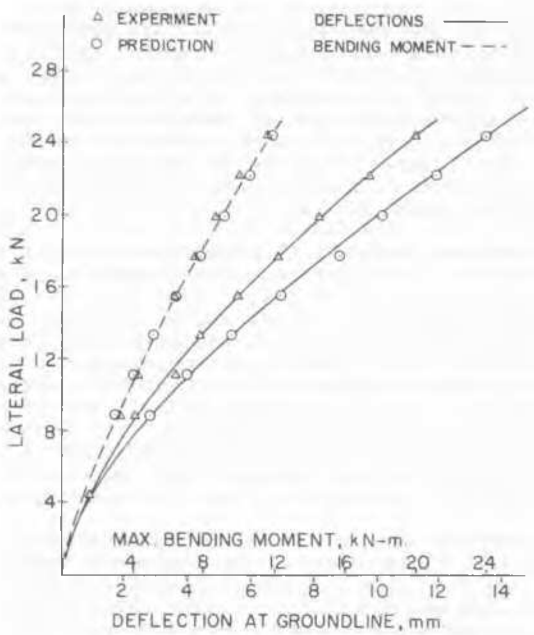

Fig. 5 Results for 152-mm diameter pile, tenth load cycle

CONCLUSIONS

In the laboratory tests relatively good agreement was found at small loads between measured deflection and deflections computed using homogeneous p-y criteria. There appears to be no consistent trend in the deflection comparisons at larger loads as the depth of the stiff layer changes.

In the field tests, excellent agreement was found between measured and computed values of maximum bending moment; the computed deflections were somewhat greater than those measured in the experiments.

The relatively good agreement between results from theory and those from experiment, or the conservatism in some of the results, tend to support the reasonableness of the analysis of laterally loaded piles using p-y curves on the assumption of homogeneous conditions. There is a need for more study of this problem, however. Additional analyses are currently underway of the results of these tests programs and it is hoped that more experimental data will become available from tests of piles under lateral load in layered soils.

ACKNOWLEDGMENT

The financial support of this work by the National Science Foundation through Grant Number ENG 74-19444 is acknowledged.

REFERENCES

Bhushan, K., Haley, S.C., and Fong, P. (1979). Lateral load tests on drilled piers in stiff clay. ASCE J GT8, 105, pp. 969-985.

Davisson, M.T., and Gill, H.L. (1963). Laterally loaded piles in a layered soil system. ASCE J SM3, 89, pp. 63-94.

Dordi, C.M. (1977). Horizontally loaded piles in layered soils. Proc., Sp. Session 10, 9th Int. Conf. Soil Mech. Found. Engg, pp. 65-70, Tokyo.

Gill, H.L., and Demars, K.R. (1970). Displacement of laterally loaded structures in nonlinearly responsive soil. Report R-670, Naval Civil Engg. Lab., Port Hueneme, Calif.

Isamael, N.F. and Klym, T.S. (1978). Behavior of rigid piers in layered cohesive soils. ASCE J GT8, 104, pp. 1061-1074.

Khadilkar, B.S., Chandrasekaran, V.S. and Rizvi, I.A. (1973). Analysis of laterally loaded piles in two-layered soils. Proc. 8th Int. Conf. Soil Mech. Found. Engg, (2.1), pp. 155-158, Moscow.

Lee, P.Y. and Gilbert L.W. (1979). Behavior of laterally loaded piles in very soft clay. Proc. 11th OTC, No. 3401, pp. 387-395, Houston.

Matlock, H. (1970). Correlations for design of laterally loaded piles in soft clays. Proc. 2 OTC, No. 1204, pp. 578-588, Houston.

Meyer, B.J. (1980). Analysis of single piles under lateral loading. Unpublished MS thesis, Uuniversity of Texas at Austin.

Naik, T.R. and Peyrot, A. (1976). Analysis and design of laterally loaded piles and caissons in a layered soil system, Methods of Structural Analysis. Proc. ASCE Nat. Struc. Engg. Conf. (2), pp. 589-606, Madison, Wisc.

Reese, L.C. (1977). Laterally loaded piles: program documentation. ASCE J GT4, 103, pp. 287-305.

Reese, L.C., Cox, W.R. and Koop, F.D. (1974). Analysis of laterally loaded piles in sand. Proc. 6th OTC, No.2080, pp. 473-483, Houston.

Reese, L.C., Cox, W.R. and Koop, F.D. (1975). Field testing and analysis of laterally loaded piles in stiff clay. Proc. 7th OTC, No.2312, pp. 671-690, Houston.

Stevens , J.B. and Audibert, J.M.E. (1979). Reexamination of p-y curve formations. Proc. 11th OTC, No.3402, pp. 397-403, Houston.

Sullivan, W.R. (1977). Development and evaluation of a unified method for the analysis of laterally loaded piles in clay. Unpublished MS thesis, University of Texas at Austin.

Sullivan, W.R., Reese, L.C. and Fenske, C.W. (1979). Unified method for analysis of laterally loaded piles in clay. Proc. Num. Methods in Offshore Piling, Inst. Civil Engg., pp. 107-118, London.

Welch, R.C. and Reese, L.C. (1975). Lateral loading of deep foundations in stiff clay. ASCE J GT7, 101, pp. 633-649.