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PREDICTION OF PILE CAPACITY

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ABSTRACT: The method used for calculating pile capacity is based on the effective stress approach applying beta and N_t-coefficients to determine the shaft and toe resistances. The capacities of the two driven piles, the pipe pile and the H-pile, are assumed to be about equal and amount to 900 KN (100 tons) on the forthcoming third testing occasion. Residual load is calculated and its influence on the load distribution is shown. Capacities at the first and second loading tests are 80 % and 90 % of that of the third occasion. The capacities of the two bored piles are expected to be about similar to that of the driven piles.

INTRODUCTION

Prediction of foundation behavior is a daily task of the foundation engineering profession. However, only rarely are the predictions compared with measurements of the actual behavior of the foundations. The academic profession, on the other hand, often gets to measure the behavior of foundation units in laboratory model scale and, sometimes, in half scale test but rarely in full scale field tests. In additional contrast and not very much realized, the academic profession seldom makes predictions, but concentrates on theoretical evaluation of known results. This situation is hidden by the erroneous use of the words "prediction" and "to predict" as synonymous to the words "calculation", "determination", "computation", and to calculate", "to determine", "to compute", etc.

On occasions, there have been interesting and worthwhile pile foundation engineering "prediction seminars". More notably, W. Lambe's seminar at MIT in 1973 and, amongst others, the FHWA's seminar in Baltimore in 1986. The first of these seminars provided data that were similar in completeness, or lack thereof, to those of a conventional engineering project. Therefore, the predictions were not very well backed up and, consequently, many predictions were off the mark. The FHWA project was arranged specially for that prediction seminar and much data were provided, but the project suffered from that it had been designed with a very limited reference to a practical engineering use of piles. Nevertheless, the prediction efforts shed light on the state-of-the-art, warts and all, as well as on the degree of applicability of sophisticated academic methods. Most important, the prediction seminars gave an opportunity to share the thrills of one's profession with colleagues and friends.

The important lesson obtained from a prediction seminar is not how a particular individual's prediction agrees with the measurements, but how the collective or average prediction is in relation to the measurements, that is, the predictive ability of the profession. It is not "who is close and who is way out of a limb?" In the context of individual performance, the only question should be "who is sticking his neck out making a prediction as a service to the profession and who is not participating?" Or, as I like to phrase it: "I expect my friends to forget that I made a poor prediction, but if I make a good one, I will not let them forget".

The present prediction effort is assisted by extensive soil, pile, and procedural information and the requested predictive effort is limited to what reasonably can be predicted at the present state-of-the-art. However, it should be understood that economical considerations and site restrictions have limited the length of the piles such that they do not correspond to what the profession would use for actual pile foundations at the site.

SOIL PROFILE

Soils investigation of the site have included standard penetration testing with split-spoon sampling and tube sampling, and laboratory classification and testing of samples; in-situ vane shear testing; static-cone penetration testing; pressuremeter testing; and dilatometer testing. The investigations have revealed two main soil strata: an upper, 7 m (23 feet) thick layer of sand overlying a thick layer of clay into which the test piles were installed a distance of 8 m (27 feet). The phreatic elevation is given as 4.5 m (15 feet) below the ground surface and the pore pressure is hydrostatically distributed.

Comments of the details of the investigation report are given below. It should be understood that my comments on the suitability of the data only refer to their quantitative use for the predictive effort, and do not reflect on the general quality of the data.

Standard Penetration Testing. The provided standard penetration test, the SPT N-indices, range between 30 through 60 indicating a compactness condition of the sand as dense to very dense. As the sand was preaugered before driving the pipe pile, its original properties are not of much importance for the predictive effort. SPT-data in the clay provide no quantitative information for direct use in the predictive effort.

Laboratory Testing. Parameters of importance for the predictive effort are the density and internal friction angle of the sand. Neither parameter is given in the information. However, using the information on natural water content of the sand given as about 20 to 25 %, a range of the total density of about 2,100 to 2,000 kg/m3 (125 to 130 pcf) is obtained for the density above and below the groundwater table, respectively. No laboratory data are given on the internal friction angle of the sand. The grain size curve indicates a very uniform fine sand (80 % in the fine-sand range). For such a soil, to assume a friction angle in the range of 35 to 38 degrees is reasonable.

The data given on the clay indicate a silty clay with an undrained shear strength of about 30 KPa (0.3 tsf), a total density of about 1,900 kg/m^3 (120 pcf). A drained, direct shear test on a sample from a depth of 12 m (40 feet) indicates a ratio between vertical stress and shear strength of 0.4. Triaxial test result indicate the somewhat smaller ratio of 0.3. The effective friction angle of the clay is not given.

In-Situ Vane Shear Testing. The undrained shear strength of the clay determined by means of a vane shear test indicates a value of about 30 to 40 KPa (0.3 to 0.4 tsf) and a sensitivity of about 2.

Static Cone Penetrometer Testing. Two types of static cones were used at the site: a conventional electrical cone penetrometer with measurement of local friction and a piezocone.

In the sand layer, the pore pressures induced by the cone can be assumed small and the data obtained can be assumed representative for the conditions during the static testing of the piles. The standard cone data show cone point resistance and local friction values of about 4,000 to 30,000 KPa (40 to 300 tsf) and 50 KPa to 220 KPa (0.5 to 2.2 tsf), respectively. These values correspond to cone resistance values which are about 100 to 200 times larger than the effective overburden stress and to local friction values of about 1 to 2 times the effective overburden stress and these ratios are very high.

The ratio between the local friction and the cone resistance is about 0.8 %, which is a higher than usual for a fine sand, in particular in a sand showing such a high cone resistance. These cone data are difficult to use with confidence for determining the pile toe resistance directly.

Because of the partial preaugering and the displacement effect of the driving, direct use of the cone data for estimating pile shaft resistance in the sand is not possible.

In the clay, neither cone provides quantitative values of direct use for the predictive effort. It is noted that the cone resistance at a depth of 15 m (50 feet) is 600 to 1,000 KPa (6 to 10 tsf). The cone rate of penetration of 20 mm/s (0.8 inch/s) is higher than the rate of movement of the pile in the static loading test. Due to this difference and because of scale effects, the pore pressure generation will be different for the cone point and the pile toe.

Pressuremeter and Dilatometer. Beyond their use for confirmation of soil layering, the pressuremeter and dilatometer data do not assist the predictive effort.

BASIC PILE DATA

Two of the four test piles are driven piles: a pipe pile and an H-pile (the pipe pile is a 450-mm (18-inch) closed-toe pile with a 9.5-mm (0.375-inch) wall and a 480-mm (19-inch) toe-plate; and the H-pile is a 355HP120 (14HP73) pile). The two other piles are 450-mm (18-inch) bored piles: one installed by means of a bentonite slurry to the full depth and one installed by means of a 24-inch casing to a depth of 9.4 m (31 feet). All four piles were installed to an embedment depth of 15.2 m (50 feet). The H-pile has two C8-18.75 channels for protection of telltales, and the pipe pile has one such channel. Each channel has a cross sectional area of 35.5 cm^2 (5.5 in^2). The nominal cross sectional steel areas of the H-pile and the pipe pile are 138 cm2 (21.4 in^2) and 134 cm^2 (20.8 in^2), respectively. The driven piles are instrumented with vibrating-wire strain gages placed at the toe of each pile and at Embedment Depths 3, 5, and 11 m (10, 23, and 36 feet). Additional instrumentation consists of telltales anchored at four depths. The bored piles seem to be instrumented in a similar manner.

METHOD OF CALCULATING PILE CAPACITY

The information includes the data from the pile driving. However, restrike data are not included and, as data from the initial driving are not applicable to capacity after soil set-up, the driving data have not been used for the predictive effort.

The soil information provided is more than one would normally have access to for an engineering design project. Yet, the information is too limited for a detailed analysis of the pile capacity. Therefore, my prediction effort is limited to the simple and straight forward effective stress analysis, wherein the shaft and toe resistances are functions of the effective overburden stress via a proportionality coefficient, beta-coefficient, for the shaft resistance and a toe bearing-capacity coefficient, N_t, for the toe resistance (Canadian Foundation Engineering Manual, 1985).

The beta-method requires as a first step the calculation of the effective overburden stress. The effective stress is a function of the density or unit weight of the soil layers and of the pore water pressure. Although the soil profile is layered, the analysis is adequately detailed by considering the soil profile to consist of two main layers, an upper, 7 m (23 feet) layer of sand fill and a lower, thick uniform layer of silty clay.

The total density of the sand layer is assumed to be 1,900 kg/m^3 (120 pcf) above the groundwater table and 2,000 kg/m^3 (125 pcf) below. The density of the clay layer is assumed to be 1,900 kg/m^3 (120 pcf). The groundwater table is assumed to be located at a depth of 4.5 m (15 feet) below the ground surface with the pore pressure hydrostatically distributed. The consequence of the potential errors in the density values is small compared to the consequence of an error in the assumption of the pore pressure distribution. Both these errors are less important than errors in the assumed beta and toe coefficients.

The beta-coefficient acting in the sand is very difficult to determine considering what effect the preaugering might have had on the pipe pile. Probably, the preaugering has reduced the shear resistance in the sand to something similar to that acting on the low-displacement H-pile. More a guesstimate than weighted judgment, the beta-coefficients assigned for use in the analysis are 0.25 and 0.45 above and below the groundwater table, respectively. As to the beta-coefficient in the silty clay, the actual value is expected to lie in the range of 0.3 through 0.4, and, again mostly as a guesstimate, a value of 0.35 is assigned. The direct shear test indicated a higher value (0.4). The slightly lower value chosen is intended to account for some pore pressure build-up and the effect of strain-softening when loading the pile.

Toe-coefficients are in general more difficult to determine than the shaft coefficients. For a toe-coefficient in silty clay, the author has in the past used the low range of N_t equal to 1 through 2. Using a value of 1.5 and considering that the effective overburden stress at a depth of 15.2 m (50 feet) is 200 KPa (2 tsf), the toe resistance becomes equal to 300 KPa (3 tsf). This is about half to a quarter of the cone point resistance. For toe resistance in clay, a common approach is to assigned it a value equal to 9 times the undrained shear strength. The undrained shear strength is slightly more than about 30 KPa (0.3 tsf), and multiplying this value with a factor of nine gives about the same toe-resistance value as the effective stress approach with $N_t = 1.5$.

PREDICTED CAPACITY

Fig. 1 shows the calculated distributions of effective stress in the soil and of the shaft resistance along the pile after full reconsolidation has occurred after the installation disturbance.

Capacity of the Two Driven Piles. The approach outlined above is applicable to the calculation of capacity of the two driven piles, the 18-inch pipe pile and the 14-inch H-pile. For the H-pile, the circumference of the box is used as the circumference of the pile. Thus, for the pipe pile, the circumference is 1.44 m (4.71 ft) and for the H-pile it is 1.42 m (4.67 ft), that is, essentially the same. Therefore, the shaft resistance is also essentially the same for the two piles.

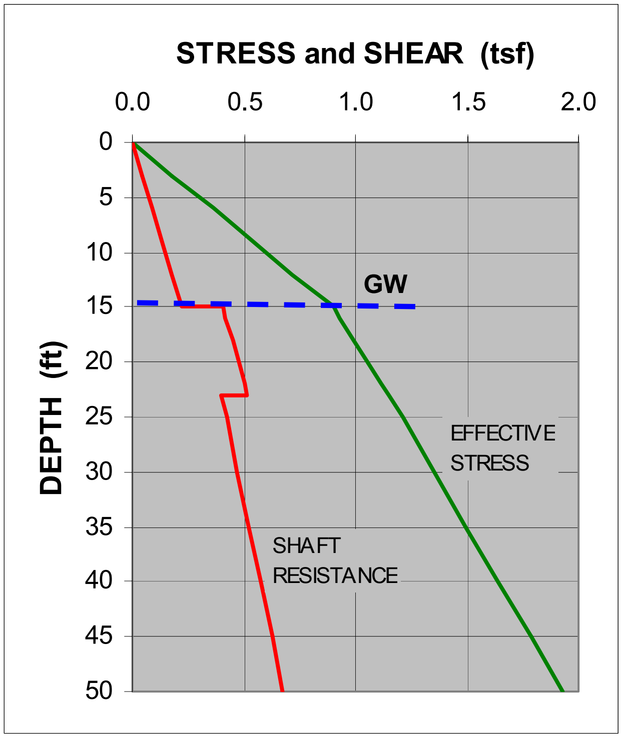


Fig. 1 Distribution of effective stress and shaft resistance.

With regard to the toe resistance, the toe area of the pipe pile is 2.0 ft^2. Because in determining the shaft resistance, it was assumed that the soil inside the flanges would plug, the H-pile toe area is also the cross sectional area of the box, that is, 1.4 ft2. (It is irrelevant whether one considers the toe resistance to occur at the end of the box or to occur as an increase of shaft resistance near the pile toe from shear within the flanges).

The geometry of the two driven piles indicates that they should test to about the same static capacity. Because the H-pile does not displace as much soil as the pipe pile, it could be assumed to carry marginally less load, but such considerations are not possible to include in the simplistic approach used for the prediction.

Calculations using the soil resistance parameters indicated above result in that the capacity of both the two driven piles is predicted to be 900 KN (100 tons). Fig. 2 shows the calculated load distribution along the piles using the shaft resistance of Fig. 1.

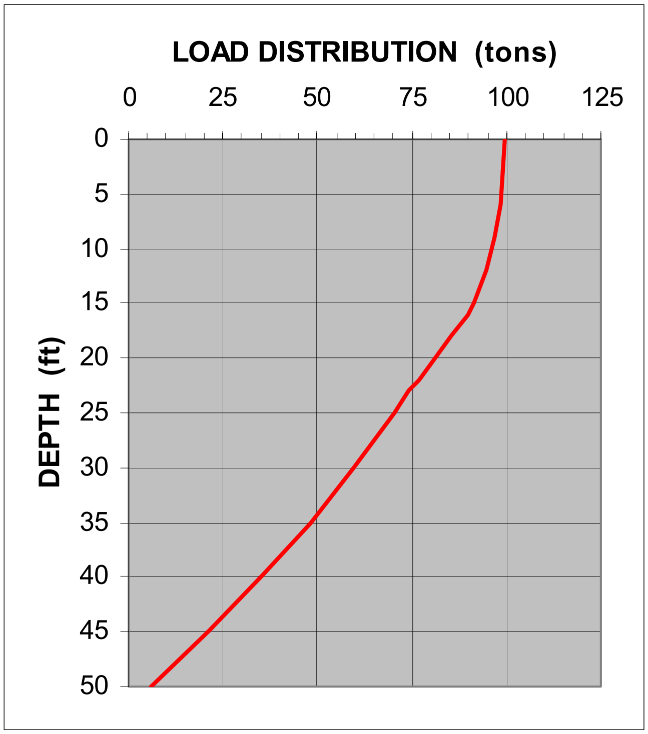


Fig. 2 Distribution of load in the pile at the final loading test (at failure)

Residual Load. The reconsolidation after driving will cause residual load to build up in the pile. These residual loads will increase for each testing occasion. For the intended third static loading test, which is the one considered in Fig. 2, the residual load (maximum values) is expected to follow the distribution shown in Fig. 3. This distribution has been obtained by construing two load distribution curves: One upper curve going from the pile head toward the pile toe determined as the integral of fully developed negative skin friction (taken as numerically equal to the positive unit shaft resistance) and one lower going from the pile toe (subjected to zero toe resistance) toward the pile head and determined as the integral of the positive shaft resistance. The intersection of the two curves defines the neutral plane for the pile above and below which the upper and lower curves define the distribution of residual load in the pile. (Near the neutral plane, the curves have been smoothed by means of a vertical portion).

It is not clear from the given information whether or not the instrumentation will allow the residual load to be considered fully, partially, or not at all. The consequence of not at all considering the residual load is indicated by the load distributions shown in Fig. 4 and labeled "uncorrected".

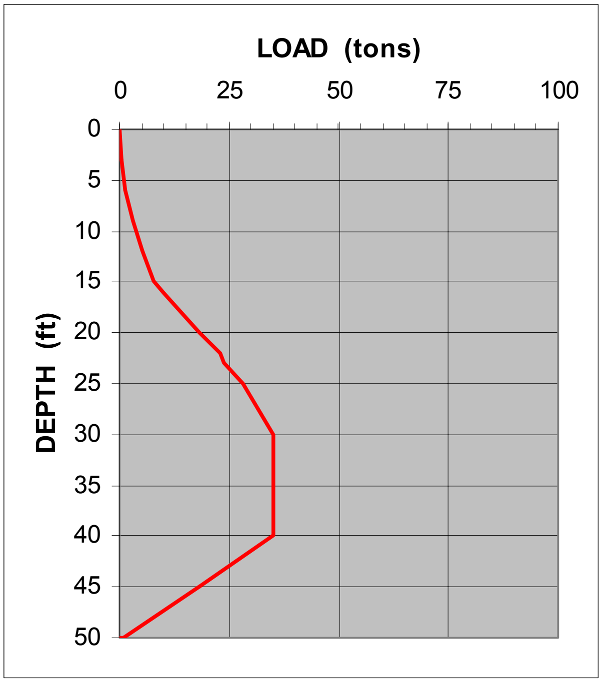


Fig. 3 Distribution of residual load

The Static Loading Test. Because the toe resistance is so small and the soil is strain-softening, the load-movement curve is expected to build up to a maximum load at a pile head movement of about 7 mm (0.3 inch) that then is constant or reduces with increased movement. The movement at the maximum load is equal to the movement necessary for mobilizing the resistance at the lower portion of the pile, about 3 mm (0.1 inch) plus the compression of the pile for the load increase over the residual load in the pile. The pipe pile and the H-pile are both assumed to test to a load-movement curve similar to the one shown in Fig. 5. That the piles have different moduli—a combined "elastic" modulus of about 50 GPa (8,000 ksi) for the pipe pile and 200 GPa (30,000 ksi) for the H-pile — will not make much difference.

Not much information has been provided about the method of testing beyond a reference to the ASTM designation for the "standard loading procedures method" (?). This may mean that only eight or ten increments would be used before failure occurs. This would be unfortunate because the rapid and rough load application associated with large load increments reduces the suitability of the test results for analysis. Therefore, it is hoped that the test is carried out using many small increments applied gently. Above all, it is hoped that no unloading will be employed in the test as this would greatly reduce the value of the test as reference to the prediction effort.

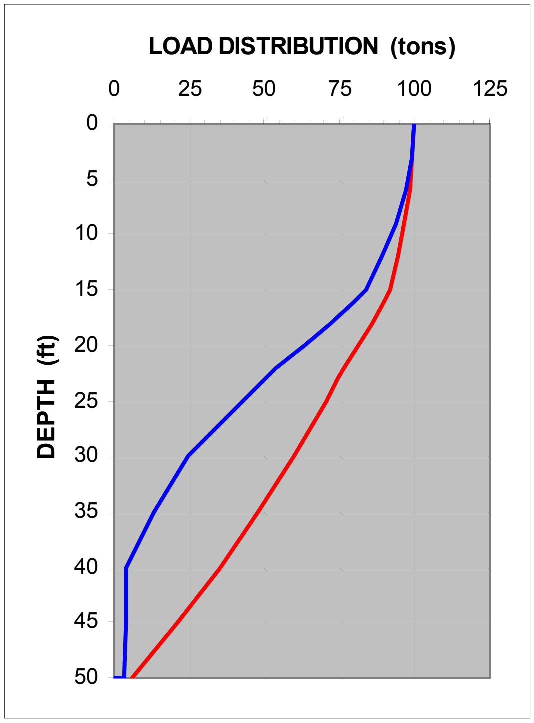


Fig. 4 Distribution of load in the pile corrected and uncorrected for residual load

It is believed that the load-movement curve and the capacity of 900 KN (100 tons) represent the behavior of the piles on the third testing occasion and that the values do not exceed the actual test capacity by more than 11 percent (the "90-percent confidence" value).

Furthermore, the capacity at the first and second testing occasions (two weeks and one month after the end of the driving, respectively) is predicted to be 80 % and 90 % of the final capacity, that is, 700 KN (80 tons) and 800 KN (90 tons), respectively. The load-movement curve for the first test is expected to be considerably flatter than the curve shown in Fig. 5.

Capacity of the Two Bored Piles. In contrast to the driven piles, the two bored piles are very difficult to analyze. For the slurried pile, the augering in the slurry and the slurrying itself can have a widely variable effect on the capacity of the pile. As to the other pile, the use of casings has a similar unpredictable effect. Without having access — making reference — to actual static loading tests on similar piles in the same general area, it is not possible to say whether or not the two bored piles will have a capacity that is larger or smaller than that of the two driven piles.

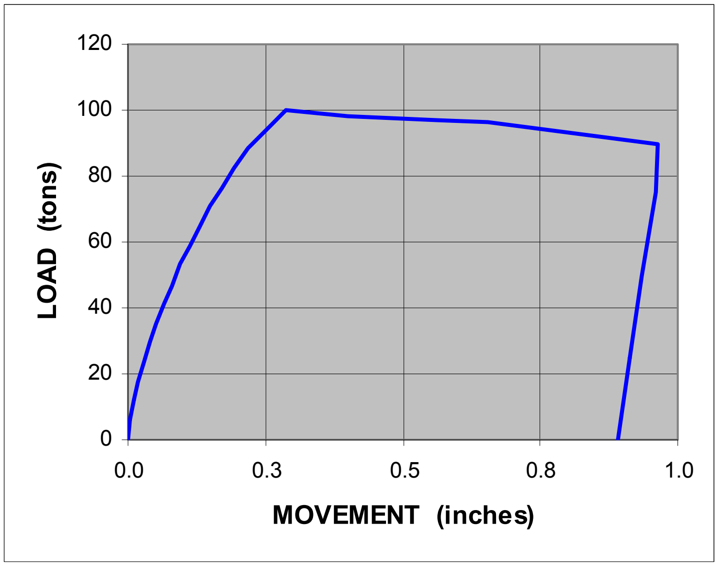


FIG. 5 Predicted load-movement curve for the third static loading test

CONCLUSIONS

The soil data given are primarily used for identifying the soil profile. Laboratory tests on density values are used directly in the analysis. Because of the dominant soil being a silty clay, for which in-situ testing data are not quantitatively applicable to the calculation of pile bearing capacity, the SPT-indices and the static cone penetrometer data have only served to assist in the selection of parameters for an effective stress approach using beta- and N_t-coefficients to determine the shaft and toe resistances.

The capacities of the two driven piles, the pipe pile and the H-pile, are assumed to be about equal and amount to 1,000 KN (100 tons) on the third and final testing occasion. Residual load is calculated and its influence on the load distribution at the ultimate load is shown. Capacities at the first and second loading tests are predicted to be 80 % and 90 % of that of the third occasion. The capacities of the two bored piles are difficult to determine and they are expected to be about similar to that of the driven piles.

The static loading test is predicted to show a sudden failure at a pile head movement of about 10 mm (0.4 inch). Thereafter, the load stays constant or reduces with increased movement.

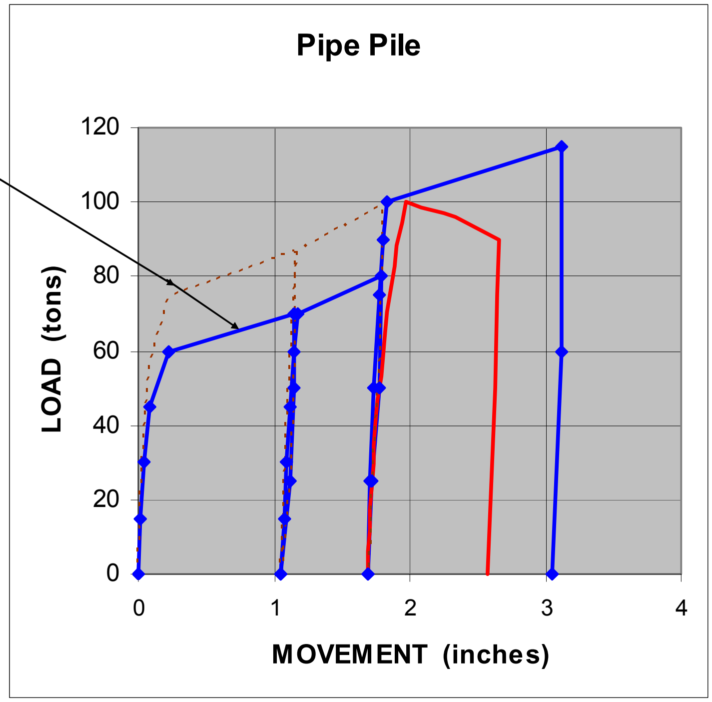
REFERENCES

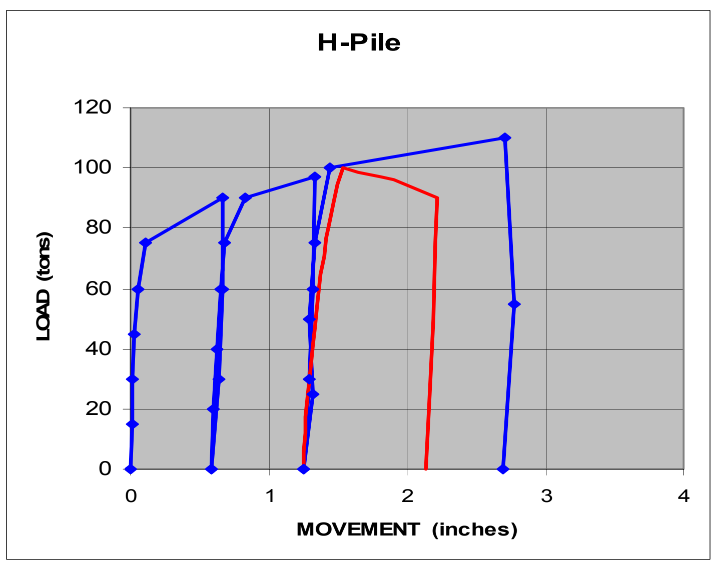
CANADIAN FOUNDATION ENGINEERING MANUAL, 1985. Part 1: Fundamentals, Part 2: Shallow Foundations, Part 3: Deep Foundations, Part 4: Excavations and Retaining Structures, Canadian Geotechnical Society, Technical Committee on Foundations, BiTech Publishers, Vancouver, 460 p.

Comparison to actual test results

The actual load-movement tests results for the two driven test piles — the Pipe Pile and the H-Pile — are presented in the following figures and the results are compared to the predicted load-movement curve.

IN CONTRAST TO THE TESTING OF THE H-PILE, THE LOADS FOR THE FIRST TWO CYCLES (2 WEEKS AND 5 WEEKS AFTER THE DRIVING) DO NOT AGREE WITH THOSE OF THE THIRD CYCLE (43 WEEKS AFTER THE DRIVING). THE DASHED CURVE INDICATES A MORE PLAUSIBLE LOAD-MOVEMENT RELATION.





For addition comparison between the prediction and the results, see Papers **"134 Choosing failure load.pdf"** and **"135 Influence of residual load.pdf"**, which are published discussions of the results.