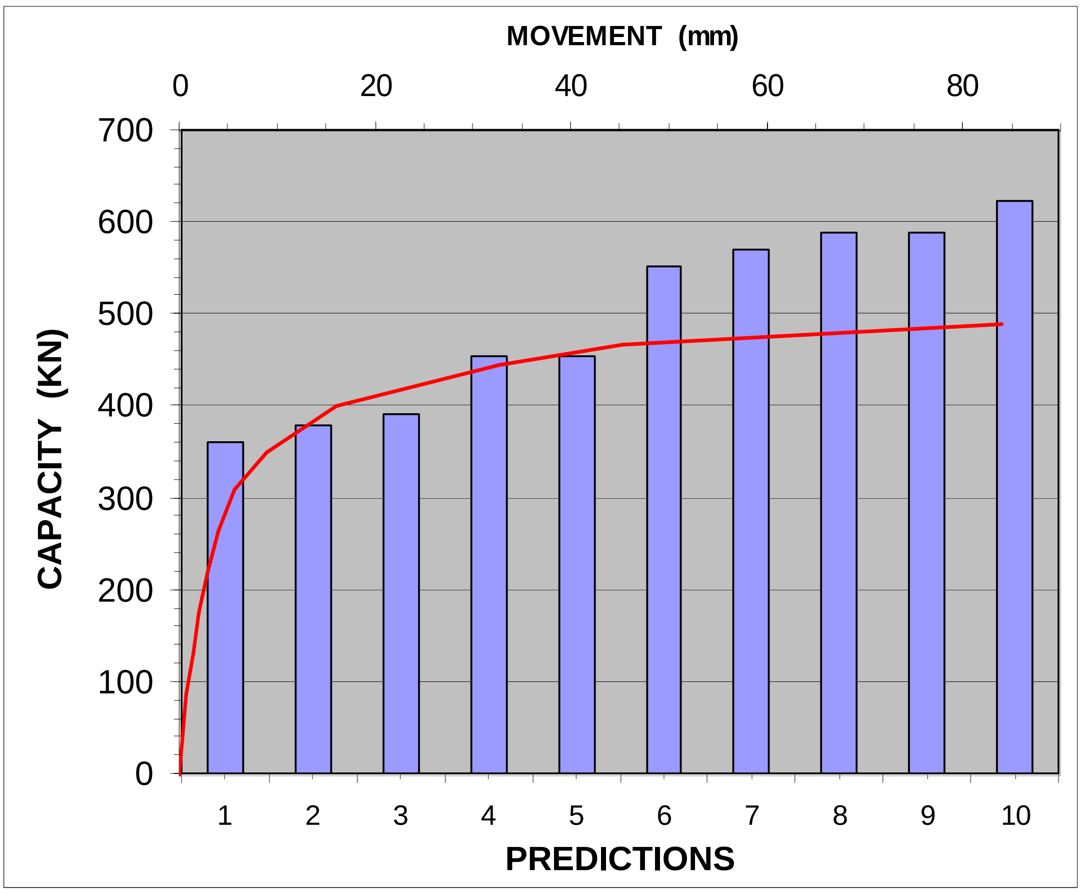
FHWA 1986 PILE PREDICTION SYMPOSIUM



Fellenius, B.H., 1986. The FHWA static testing of a single pile and a pile group—Report on the analysis of soil and installation data plus Addendum Report. The Federal Highway Administration, FHWA, Washington, Prediction Symposium, June 1986, 13 p.

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Text from 1986 has been reformatted. The available original graphics have been replotted from the original data. Added text (2006) is shown in blue.

FHWA 1986 PILE PREDICTION SYMPOSIUM — REPORT ON ANALYSIS OF DATA

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April 18, 1986

1. INTRODUCTION

1.1 Terms of Reference

The Federal Highway Administration (FHWA) is undertaking a study of bearing capacity of piles in sand. Several individuals have been contracted to predict the capacity of a specific size pipe pile based on soil information provided by the FHWA. The predictions are Class-A predictions, as they are made before the piles are installed.

The work reported herein was contracted on February 15, 1986, and is based on information on the pile and the soil as received from the FHWA between March 5 through April 7, 1986.

1.2 Objectives

FHWA's objective with the study is to generate "independent predictions of the behavior of an experimental pile group" as an example of the reliability of the present state-of-the-art to be presented at a FHWA-sponsored symposium to be held in Washington, D.C., in June 1986.

The accomplish the objective, the FHWA has requested that the behavior of a single pile and of a group of five piles shall be predicted by means of addressing the following aspects:

1. Failure mode
2. Axial bearing capacity
3. Axial load-movement
4. Shaft resistance as to magnitude and distribution
5. Distributing of load between the five piles in the group.

It is the purpose of this report to respond to the FHWA requests as much as realistically justified by the amount and, in particular, the quality of the information received.

1.3 Scope of work

The work has consisted of a compilation of the data into tables and graphics followed by an assessment of the validity of the data, a calculation of the predicted capacity of a pile as given, and writing up the work in a written report to be presented at the June symposium.

2. SOIL DATA

The site is located at Hunter's Point, Naval Shipyard Facilities, San Francisco, California.

The field information from the test site consists of the results of five boreholes with SPT-tests, three static cone penetrometer tests, two pressuremeter tests, and two dilatometer tests. Laboratory test results received consist of seven grain-size analyses and two direct-shear tests.

Below a 100 mm (4 inches) thick asphaltic concrete pavement, the soil is identified as an about 1.8 m (6 feet) thick upper layer of fill consisting of clayey sand, abundant gravel size grains, clayey sand, and concrete fragments. Below this miscellaneous fill, the soil consists of a 10.7 m (35 feet) thick layer of hydraulic fill, visually identified as sand, which was placed in 1942. At a depth of 12.5 m (41 feet), the fill is underlain by clay.

2.1 SPT-Test

Two boreholes were put down by means of rotary wash drilling in July 1985; Borehole 1 on July 5 and Borehole 2 on July 8. A third borehole, Borehole 3, was made on February 25, 1986, in connection with pressuremeter testing. Two additional boreholes, Boreholes 4 and 5, were made on March 20 and 13, 1986, in connection with dilatometer testing. The boreholes were put down at the pile test site and at a distance from each other varying from 4 m (13 feet) to 12 m (39 feet). No written report was provided.

Fig. 1 presents a diagram of the N-values given and indicates quite a scatter of values. Because of the scatter and the variable recovery, the data do not support any other interpretation than that the N-indices in the hydraulic fill range within the pile embedment depth from 10 through 20, which is representative of a compact (medium) compactness condition.

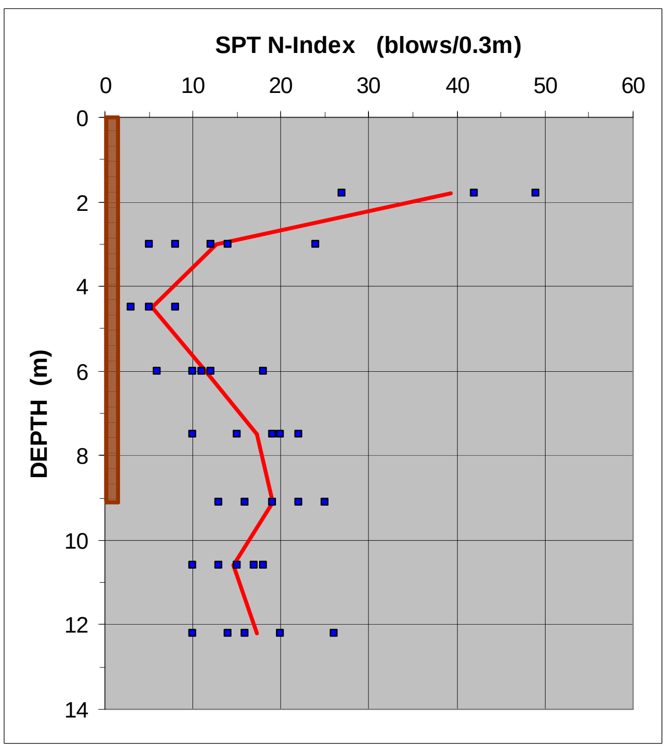

Fig. 1 Standard Penetration Test, SPT, results — N-indices

Water table observation as of July 11, 1985, is given on the borehole records indicating depth to water of 2.2 metre (7.2 feet) and 2.4 m (7.9 feet). No information is given whether or not the water observations were made in uncased or cased boreholes, a stand-pipe piezometer, or other type of piezometer, nor is any information given as to relevance to the groundwater elevation at the site.

2.2 Static Cone Penetrometer Tests

Three Static cone penetrometer tests were put down at the test site on July 8, 1985, within a distance of 6 m (20 feet) and 10 m (33 feet) from each other. The static cone penetrometer used is stated to be an electrical piezocone providing data on the cone pressure and sleeve friction when advancing the cone into the soil. No pore pressure information is included. Fig 2 shows a diagram of the results from CPT 2.

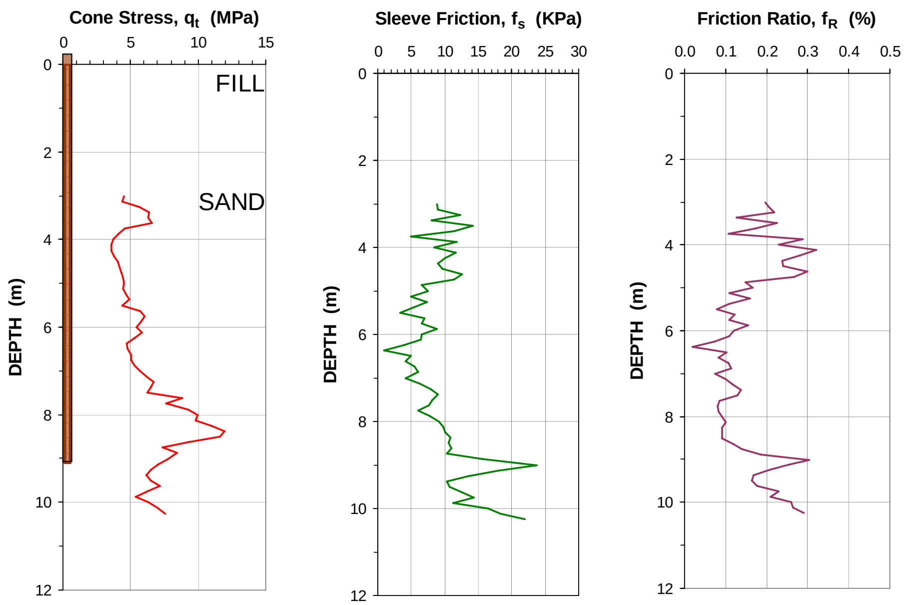

Fig. 2 Results of CPT sounding CPT2

Fig. 3 (not available) shows a diagram over the values of sleeve friction from CPT 1 through CPT3. For CPT1, the values are negative, which does not indicate a realistic penetrometer test. Therefore, all Cone 1 sleeve friction data must be discarded. The curves representing the values from Cones 2 and 3 are mostly in agreement and indicate an increase in sleeve friction with depth. Again, with a bit of good will, the increase can be interpreted to a linear development with an increase of about 1.5 KPa per metre depth. This value is very low. In naturally deposited sand, for instance, even a very loose and fine sand, at least about twice as large a value would have been expected.

Fig. 4 (not available) shows a diagram over the friction ratio values received referring to CPT2 and CPT3. The friction ratio is the ratio between the local friction and the cone stress. As evidenced in the diagram, the friction ratio in the hydraulic fill ranges, approximately, from 0.1 % through 0.3 %. This range is very low. For instance, Begeman (1965) and, also, Schmertmann (1977) indicated for silty sand, fine sand, and coarse sand ratios of 2 %, 1.5 %, and 1.2 %, respectively. Obviously, the friction ratios provided at the test site do not agree with the simple classification as sand. That the soil at the site is a hydraulic fill and not a natural deposit is probably the reason for this.

Fig. 5 (not available) shows a diagram of pore pressure values received. The values are taken during advancement of the probe. No reference to the static pore water pressure was attached with the penetrometer data, which is unfortunate. The values from Cones 2 and 3 show some agreement in the lower portion, whereas Cone 1 deviates considerably from the other two. Cone 1 pore pressure values are considered unreliable.

2.3 Pressuremeter Tests

On February 26 and 27, 1986, a total of 9 preboring and 4 selfboring pressuremeter tests were conducted at the test site. The preboring tests were made in Borehole 3. The test results were given in detail in a report by Briaud and Tucker for Geo/Resource Consultants and dated March 1986. The report uses a mixed system of units: pressure and modulus in Pa and depth in feet.

As shown in Fig. 6 (not available), the Net Limit Pressure increased with depth in the hydraulic fill. The increase was approximately linear from a value of about 300 KPa at Depth 3 m to 900 KPa at Depth 12 m, i.e., about 70 KPa/m. According to the Canadian Foundation Engineering Manual (1985), the range of values is typical of a loose silty sand.

As shown in Fig. 7 (not available), also the Pressuremeter Modulus increased approximately linearly with depth from a value of about 2.5 MPa at Depth 3 m to about 6.5 MPa at Depth 12 m, i.e., about 400 KPa/m.

Because of the approximate linearity of the Net Limit Pressure and the Pressuremeter Modulus, the ratio between the values is approximately constant and about equal to 6. According to the Canadian Foundation Engineering Manual (1985), this ratio is typical of a loose silty sand.

The report by Briaud and Tucker provides also values of the earth pressure at rest, K_0, which is indicated to be approximately constant in the hydraulic fill and about 0.9. The values of K_0 were computed using an empirical formula, which includes the total unit weight of the soil. However, the unit weight of the soil was not given. By back-calculating from the values of K_0, it is clear that the report has used a constant value of the unit weight of the soil equal to 19.3 KN/m^3 (120 psf). As no value of the unit weight of the soil is found anywhere in the documentation distributed by the FHWA, it is probable that the unit weight applied for the calculation of K_0 was assumed. If, instead, a unit weight of 17 KN/m^3 (106 psf) would be used, values of K_0 would result which are about 25 % larger than those given in the report.

2.4 Dilatometer Tests

Two dilatometer tests were carried out at the test site in March 1986. The results were presented in a report by R.L. Handy to Geo/Resource Consultants Inc. and dated March 24, 1986. The report provides results of the tests in terms of K_0 and internal effective friction angle of the hydraulic fill. In one borehole, the values of K_0 ranged from 0.5 through 0.6 and the friction angle values range from 28° through 35°. In the second test, the values of K_0 ranged from 0.6 to 1.0 and the friction angle from 27° through 38°. The values were evaluated from several formulae based on simplified assumptions and using "inferred densities". The report includes indirect warnings as to the reliability of the data, for instance, "*Surprisingly, the data aren't too bad*", and "... *a number of parameters are derived, the accuracy of course depending of the reliability of the supporting equations*".

For the purpose of a prudent engineering design, the dilatometer tests results are too variable and the reliability too questionable to be used. Consequently, there is little purpose in applying them in a prediction effort.

2.5 Soil Parameters and Laboratory Data

The documents received from the FWHA do not include information on measured density or unit weight of the soil. The pressuremeter report uses an assumed unit weight of 19 KN/m^3 (120 pcf) and the dilatometer report mentions "*inferred densities*", but the reports do not quantify the values. Further, the documents do not contain any clear indication of where the groundwater table is and if the pore pressure distribution is hydrostatic or if there is a gradient in the soil. Presumably, the site is close to a body of water. However, there is no mentioning of this, nor are general water elevations given or tidal action indicated.

There is no mentioning of the general geology of the area, nor of the geological origin and mineralogical composition of the hydraulic fill.

Seven grain-size curves were received indicating that the hydraulic fill consists of 70 % medium sand, 15 % coarse sand, and 15 % fine sand as per grain-size classification according to the Canadian Foundation Engineering Manual (1985). The coefficient of uniformity is about 2, which classifies the soil as very poorly graded (Holtz and Kovacs, 1981). The grain size curves received have been compiled in Fig. 3. The grain-size range is similar to that shown in a compilation of grain size curves from a pile testing programme in naturally deposited, medium size sand at St. Charles River, Quebec, reported by Tavenas (1971). Tavenas (1971) reported that the bulk unit weight of the soil was determined in-situ to be 15.4 KN/m^3 (98 pcf) and that the SPT N-indices ranged from 19 through 31.

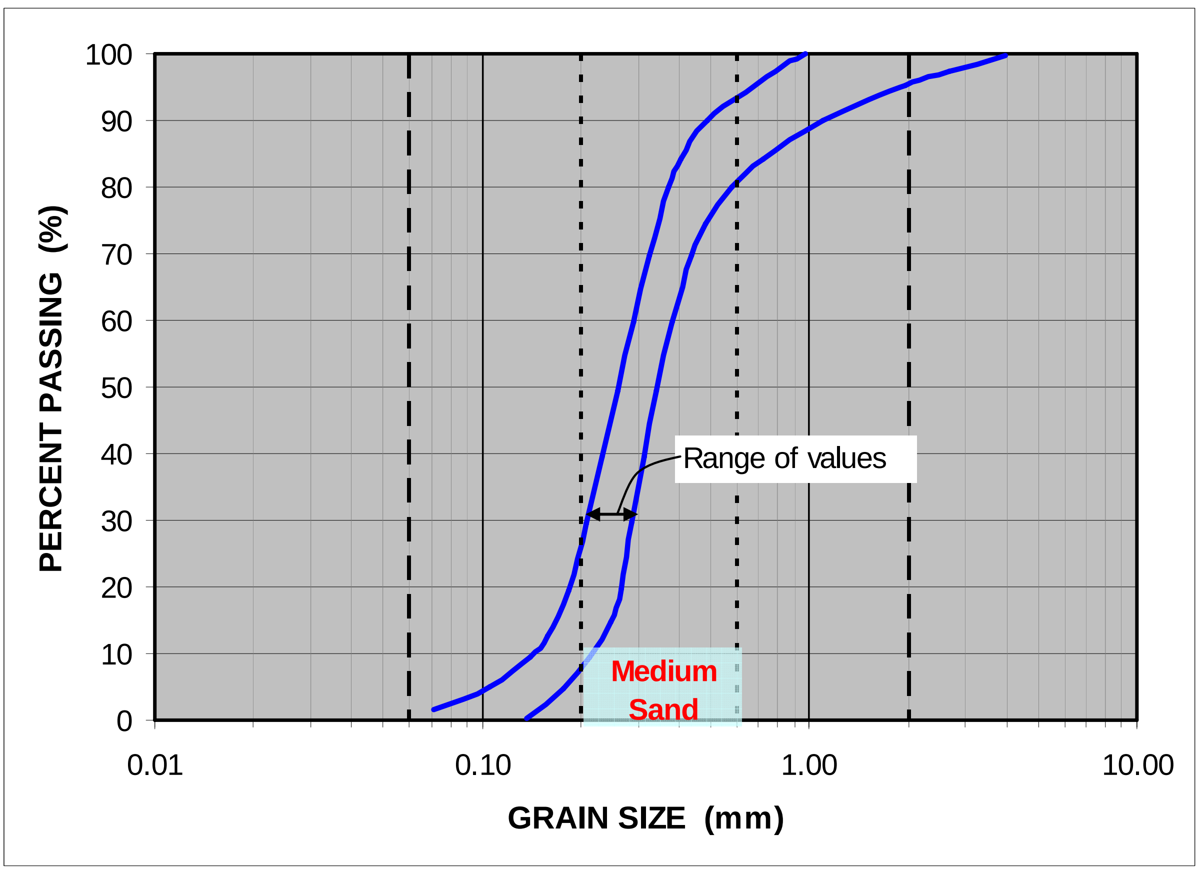

Fig. 3 Grain-size curves

The FHWA provided two rough sketches showing results of five direct shear tests on samples from BH 3 at Depths 3.5 m (11 feet) and 9.5 m (31 feet). The sketches are in the form of shearing-stress versus normal-pressure plots and indicate the result of a test as dots on a line sloping from the origin. The two lines slope 34° and 36°, respectively. Thus, it is inferred that the friction angle of the soil sample tested is about 35°. However, lacking all information on density, sample preparation, etc., the representativeness of the tests is in question.

3. PILE DATA

The test pile is stated to be a closed-toe, 273 mm (10.75 inches) O.D. steel pipe with 9.5 mm (0.375 inch) wall thickness. The nominal cross sectional area of the steel is 78.9 cm^2 (12.23 in^2) and the nominal circumferential or perimeter area is 0.858 m^2/m (2.81 ft^2/ft). The gross cross sectional area is 824 cm^2 (90.8 in^2).

The data do not contain any details on the specific type of pile used nor any verification measurement of the size of the pile used for the test piles. Presumably, the pipes will follow ASTM D252 Standard. The ASTM standard allows a deviation of weight of 15 % over and 5 % under that specified are taken as the potential deviation of the cross sectional. As to the deviation of outside diameter, only 1 % deviation is allowed, which indicates the potential deviation of the perimeter area. The pile is installed closed-toe. The size of the toe plate is not indicated. However, it is assumed that it is flush with the outside diameter of the pipe.

The embedded length of the pile is 9.1 m (30 feet). The stick-up appears to be 0.9 m (3 ft).

The piles will be instrumented with seven levels of strain gages. However, the actual levels are not included in the information given. In addition to the strain gages, one telltale is installed to the toe, where also a pair of total stress and pore pressure cells are placed. The telltale is placed in a plastic guide-pipe. It is therefore assumed that the guide-pipe will have no influence on the stiffness of the pile.

The piles will not be concrete filled. There is no information provided as to the arrangement of the reaction load for the test. However, the requirement for available reaction load is set to 2,000 KN (200 tons) per pile.

There will be six test piles, one single pile and a group of five piles placed in a square configuration with four corner piles and one center pile at a c/c distance from the corner piles of three pile diameters, i.e., 0.82 m (2.7 feet). The pile group will be tested as a unit.

Of the group piles, the center pile will be driven first. There is no information on whether or not the piles will be restruck after driving.

There is no information provided indicating the method of loading to be applied in the static test, for instance, whether maintained load or constant rate of penetration methods, and, if a maintained load method, what number of increments to be used and whether a slow or a quick test will be applied. The report states that the magnitude of the load increments is chosen small enough to ensure that no fewer than about 25 load increments will be used in the loading tests even if the ultimate resistance would show to be smaller than 2,000 KN (200 tons). Therefore, in case of testing a single pile, load increments may have to be chosen smaller than 50 KN (5 tons). Consequently, as the accuracy of the applied load needs to be better than about 5% of the increment value, an accuracy of about 2 KN (0.2 ton) is desirable.

4. COMMENTS ON THE DATA PROVIDED

The capacity prediction of the test piles at the site must be based solely on the soil information provided before the installation of the piles. Presumably, an objective is that the prediction effort be useful as reference to the design of actual piling projects in engineering practice. However, the design of the project, as well as the quality of the information provided make this objective difficult to achieve.

The size of the piles corresponds to a size which is commonly used for piling. It is commendable that a full-scale test was planned for this predictive effort. However, it is not likely that this size pile would be used for an actual engineering project in the soil found at the test site. In an engineering project, this soil would be bypassed with the piles and the foundation be taken deeper and into more competent soil. Alternatively, other size and types of piles would be more economical for the site, or other methods would be chosen for the foundation at the site, for instance, dynamic compaction of the sand.

More important, had a real project yet been slated for a piling foundation in the hydraulic fill, no experienced engineer would have put his professional liability on the line by committing himself to a design without investigating the compactive effect of pile driving at the site. In all likelihood, the very uniform, medium sand is sensitive to vibration, maybe even collapsible, and driving piles will have a pronounced compactive effect. Thus, the properties, such as density, internal friction angle, and, therefore, bearing capacity will be manifestly affected by the pile driving.

Furthermore, important data are lacking in the documentation provided to the predictors, such as information on the in-situ density, but also the lack of results of laboratory testing at different densities makes it impossible to unearth an effective internal friction angle as well as to arrive at a more logical estimation as to the compactive effect of the pile driving.

The predictors are requested to provide an analysis of the group effect of the piles. Well, suppose that the soil underneath a pile was compacted when the pile was driven. Then, when the surrounding piles are driven, the soil underneath the toe of first pile may loosen, which would reduce the capacity of the pile. Or, on the other hand, suppose that the driving of the other piles compacts the soil around the shaft of the first driven pile dragging it down into a compacted sand below the pile toes, which will increase the capacity. Both possibilities could occur simultaneously at the test site, but, unfortunately, it is not possible to predict which pile will be subjected to what condition in this regard. As the prediction effort is not an engineering project where professional liability concerns exist, a prediction can and will be made.

5. ANALYSIS

5.1 General

SPT-Test

The information provided lends itself to a calculation using the results of the N-indices from the SPT-tests according to Meyerhof (1976) as quoted in the Canadian Foundation Engineering Manual (1985).

(5.1) $$R_u = R_t + R_s = m\,N\,A_t = n\,N_{avg}\,D\,A_s$$

where
R_u = ultimate resistance
R_t = toe resistance
Rs = shaft resistance
A_t = area of pile at toe
A_s = circumferential area
m = empirical coefficient, $400\ 10^3$
N = N-index at pile toe
N_{avg} = average N-index along pile
D = embedment depth

All units are base SI-units

Static Cone Penetrometer Test
Ordinarily, the unit toe resistance is determined from the static cone penetrometer results as a weighted average of cone stress values near the pile toe (Schmertmann, 1988). The shaft resistance is most directly determined by putting it equal to the value of the local friction.

Pressuremeter Test
Use of results from pressuremeter tests to determine capacity of piles in loose soil is not well established. It has been suggested that the shaft resistance should be taken as about 10 % of the limit pressure, but not greater than a certain value. However, the only value of the pressuremeter tests for the subject prediction effort lies in that it gives another indication that the soil is fine-grained and not very competent. The pressuremeter data will not be used in the prediction.

Geotechnical Formula
It is well established that static resistance of pile sis proportional to the effective stress in the soil. The geotechnical formula requires knowledge of the effective overburden stress in the soil, i.e., the total unit weight and the pore pressure distribution, as well as the effective internal friction angle of the soil. Sometimes, a Critical Depth is applied to fit the analysis to actually measured values. The Critical Depth is a depth below which the calculated shaft and toe resistances are assumed to become functions of constant values of effective overburden stress. Use of the Critical Depth is questionable, however, as it is an empirical approach to solve an apparent lack of progressive increase of resistance with depth, which can be adequately explained by considering the presence of residual strain in the pile. As the residual strain will be measured in the static loading tests, the Critical Depth concept is not used in the prediction.

5.2 Soil Parameters and Soil Data

The soil unit weight is not known. However, based on the fact that it is an hydraulic fill consisting of very uniform medium sand, a unit weight of 16 KN/m^3 (100 pcf) is reasonable, as found for the similar sand by Tavenas (1971) and this value will be applied. In the 1.8 m thick, miscellaneous upper fill layer, a unit weight of 19 KN/m^3 (120 pcf) will be applied.

The groundwater table and pore pressure distribution is not known. However, it is more than reasonable to assume that the observation of a water table in the boreholes at Depth 2.2 m (7.2 feet) corresponds to the groundwater table and that is hydrostatically distributed in the sand.

The effective internal friction angle is not known. A compact uniform sand such as the hydraulic fill can be expected to have a friction angle of at least about 34°. However, the sand will displace and compact during the pile driving and, therefore, the friction angle will increase. Kishida (1967) suggested that the increase in sand below the pile toe is equal to half the difference between the original friction angle and 40°, which in this case amount to about 34°. Applying this value, the final friction angle at the pile toe would become about 37°. For the shaft resistance, there is no such simple relation, however.

The mentioned unit weights and pore pressure conditions result in an effective overburden pressure increasing by 6 KPa per metre depth in the hydraulic fill. A comparison of this value to the increase of cone stress, which was established to be 1,000 KPa per metre depth, indicates a cone N_t-coefficient of about 160. Should the relation $N_t = 3N_q$ be valid, an N_q-coefficient of about 50 and a friction angle of 38^o result, which values are reasonable, but somewhat on the high side. The relation proposed by the Canadian Foundation Engineering Manual (1985) between N_t and N_q of 3 is, however, very questionable.

At Depth 9.1 m, the effective overburden stress is about 90 KPa and the cone stress is about 8 MPa. This indicates a N_t-coefficient of about 90, which is more reasonable than a value of 160, but still judged to be on the high side. In the prediction analysis, a N_t-coefficient of 80, that is, at Depth 9.1 m, a unit toe resistance of 7,000 KPa will be applied. (***In a 20-year hindsight, this is way way too high a value!***).

The unit shaft resistance increases with the increase in effective stress. Thus, the increase in effective stress of 6 KPa per metre depth compared to the increase in local friction of 1.5 KPa established for the static cone penetrometer tests suggests a ß-coefficient of about 0.2. This value would be low in a soft clay, for a compact sand, it is judged to be much too low. Considering compaction effect from driving a single pile, a beta-coefficient of 0.4 is suggested for use in the prediction. In the upper fill, a beta-coefficient of 0.6 is suggested. (***A UniPile calculation using the mentioned unit weights and coefficients results in a total shaft resistance of 176 KN and toe resistance of 418 KN; Pile capacity of 594 KN***).

The St.Charles River testing programme reported by Tavenas (1971) involved static test loading of instrumented 0.3 m (12 inches) precast concrete piles. For a pile driven to a depth of 10 m (30 feet) into the medium sand, Tavenas (1971) reported an average unit shaft resistance of 24 KPa (0.25 tsf) and a toe resistance of 4,200 KPa (44 tsf)

5.3 Analysis from SPT-Tests

The average SPT N-index along the pile shaft is about 15. Therefore, Eq. 5.1 indicates that the ultimate resistance of the pile is:

$$R_u \quad = \quad 400 \cdot 15 \cdot 0.0585 \; + \; 2 \cdot 15 \cdot 9.1 \cdot 0.858 \quad = \quad 351 \; + \; 232 \quad = \mathbf{583\ KN\ \ (131\ kips)}$$

The calculated value does not consider any compaction effect from pile driving. Furthermore, it should be recognized that the relation according to Eq. 5.1 is empirical and has not been derived from piles driven in hydraulic fill of any kind. Consequently, not much weight should be given to this calculation.

5.4 Analysis from Geotechnical Formula supported by Cone Penetrometer Tests

The calculations are presented in the following table

Depth	Unit weight	Total stress	Pore pressure	Effective stress	Beta coefficient	Unit shaft resistance
0	19	0	0	0	0.6	0
1.8		34.2	0	34.2		20.5
1.8	17	34.2	0	34.2	0.4	13.6
2.2		41.0	0	41.0		16.4
9.1		158.3	69.0	89.3		35.7

Although the text indicates a unit weight of 16 KN/m^3, the above calculations use 17 KN/m^3. A careless mistake, resulting in a capacity value than is 10 KN higher than the intended value..

The effective stress analysis results in the following values.

R_s = 0.858·0.5[20.5·1.8 + 0.4(13.6 + 16.4) + 6.9(16.4 + 35.7)] = 175 KN

R_t = 0.0585·7,000 = 410 KN

R_u = 175 + 410 = **585 KN)**

The agreement with the SPT-method is purely coincidental.

The average unit shaft resistance in the hydraulic fill below the water table is 26 KPa (0.25 tsf), which is practically the same as found by Tavenas (1971). The unit toe resistance of 7,000 KPa is considerably larger than the 4,200 KPa found by Tavenas (1971).

The prediction of ultimate load of 585 KN (66 tons) is considered valid for the single pile. If a validity range of capacity would be given, the range would be judged to be -100 KN (20 kips) and +300 KN (60 kips). This unbalanced range is considered to include a margin for strength increase due to compaction of the sand around and below the pile. Definitely, should the group piles be restruck, the capacity of the piles will increase and, probably, exceed 800 KN (180 kips). Although the soils is sand, the possibility of a time dependent increase of capacity cannot be excluded. As stated earlier, due to the limited information, it is not possible to predict reliably the capacity nor the load-deformation behavior of the piles in the pile group. It is simply assumed equal to five time that of the single pile.

A residual load will develop in the pile during and immediately following the driving. This load is considered in the above analysis and is expected to be about 100 KN (20 kips) acting at a neutral plane at a depth of about 7 m (22 feet) and mainly resisted by toe resistance. (***The prediction figures indicated a predicted residual toe load of 60 KN***)

The ultimate load will be reached at a small toe movement, approximately 15 mm (0.6 inch). There will be a small compression of the pile shaft.

6. REFERENCES

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A D D E N D U M JUNE 14, 1986

Written a week after the results on the loading test on the single pile had been presented

A1. WAVE EQUATION ANALYSIS

Ten days after the prediction analysis was completed, on April 30, 1986, between 08.20h and 13.40h, the six test piles were driven at the site to the predetermined toe depth of 9.1 m (30 feet). The pile driving hammer was a Delmag D22 single acting diesel hammer. According to earlier information from the FHWA, the pile driven first was the single pile, the center pile in the group was the second pile driven, and the four corner piles were driven last. The piling log, as available to the Author on May 20, 1986, has been compiled in the following table.

PILE DRIVING LOG

Pile	Penetration Resistance for last 0.3 m (blows/0.3 m) (blows/foot)	Accumulated Number of blows below 1.5 m (5 feet) (blows)
Single Pile	6	62
Center Pile	7	62
1st Corner Pile	8	100
2nd Corner Pile	9	129
3rd Corner Pile	9	9l
4th Corner Pile	11	122

The driving data indicate a penetration resistance (PRES) increasing with depth. The PRES at termination of the driving of the single pile and the center pile was 6 and 7 blows/ foot, respectively. For the corner piles, the PRES varied from 8 through 11 blows/foot, which appears to indicate a slight effect of densification of the soil from the driving of the center pile. The total numbers of blows used to drive a pile seems to indicate this more clearly; the two first driven piles (center pile and corner pile) required a total of 62 blows each, while the remaining three corner piles required 91 to 129 blows.

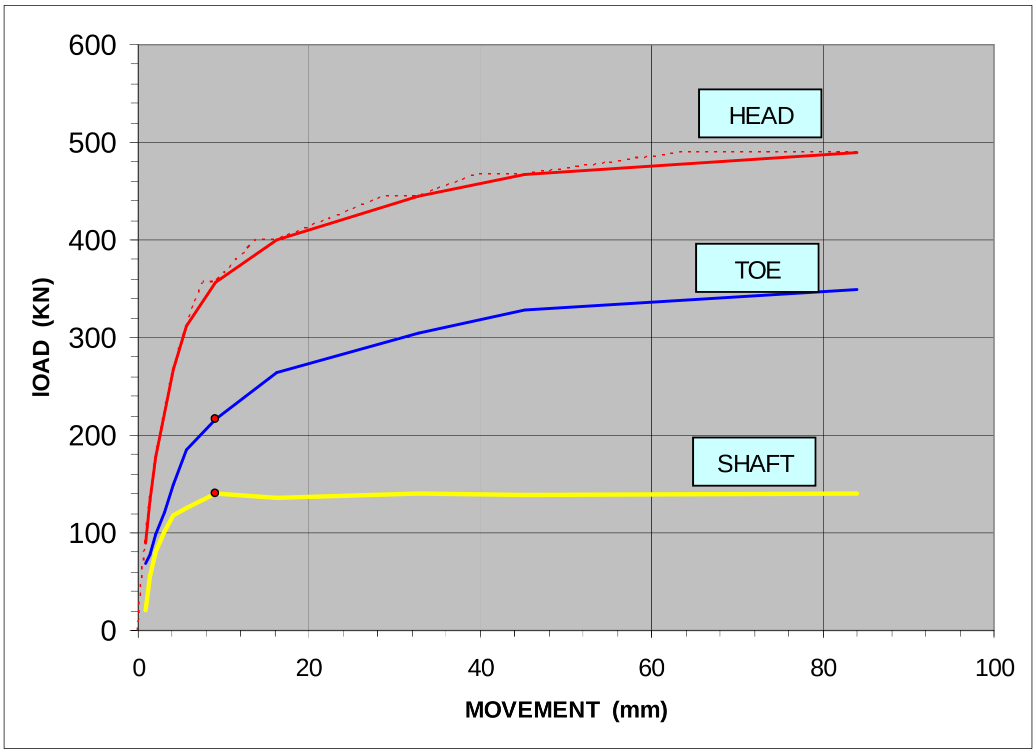
A wave equation analysis using CUWEAP was made on the available data. The results are: At PRES of about 7 blows/foot, the Rult indicated is about 360 KN (80 kips). At a PRES of 10 blows/foot, the RULT is 540 KN through 630 KN (120 kips through 140 kips). Thus, the seemingly slight densification effect may yet have had a significant effect on the pile capacity.

It must be considered, however, that the densification evidenced in increased PRES when driving one pile will loosen the soil around the previously driven pile, in particular around the toe. Therefore, an actual increased capacity may only be observed for the corner pile driven last, while the center pile may demonstrate a smaller toe resistance than the single pile. It is predicted that the measurements of residual compression strain (locked-in strain prior to the test) will be larger for the center pile than for the other piles.

A2. STATIC TESTING OF THE SINGLE PILE

Preliminary results of a static loading test on the single pile carried out on May 23, 1986, were received on June 12, 1986. The loading test was carried out as a maintained-load test with ten increments of 10 kips (4.45 KN) followed by two increments of 5 kips (2.25 KN) every about 0.5 hour. The total load applied was 109 kips (485 KN).

The following diagram plotted from the received data presents the load-movement curves of the test.



Load-movement curves from static loading test on the single pile.
The toe and shaft curves include correction for residual load

Judging simply from the shape of the curve, the ultimate load, R_u, ranges from 445 KN through 490 KN (100 kips through 110 kips). The Davisson offset limit is 350 KN.

COMPARISON TO ACTUAL TEST DATA

	R_u (KN)	R_s (KN)	R_t (KN)	Mvmnt (mm)
Author's prediction 1986	587	178	409	15
Test, Offset Limit	350	140	210	8
Test, 15 mm movement	400	140	260	15
Test, 10 % of diameter	431	140	291	27
Test, Plunging	485	140	345	83

The measured residual toe load was 61 KN. The author predicted 60 KN.

The neutral plane (force equilibrium) was between 7.0 m and 7.5 m.

Employing the more reasonable unit density of the hydraulic sand of 15.5 KN/m^3 or 16 KN/m^3, the shaft resistance, 140 KN, indicated at the Offset Limit corresponds to ß-coefficients of 0.4 in the fill, 0.3 to 6 m depth in the hydraulic sand and 0.45 below this depth. The predicted shaft resistance overestimates the actual by 38 KN or 27 %.

The toe resistance, 210 KN, indicated at the Offset Limit corresponds to an N_t-coefficient of 45. The predicted toe resistance is clearly off.