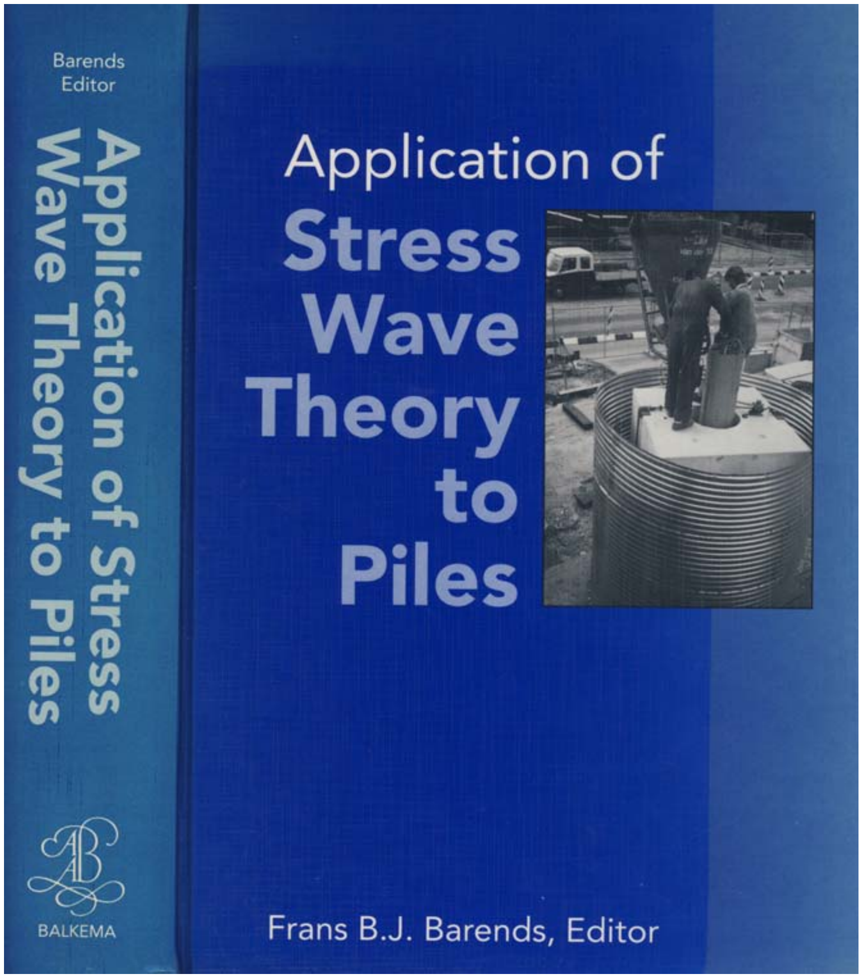

FELLENIUS, B. H., EDDE, R. D. and BERIAULT, L. L, 1992. Dynamic and static testing for pile capacity in a fine-grained soil. Proceedings of the Fourth International Conference on the Application of Stress-Wave Theory to Piles, Ed. F.B.J. Barends, DenHague, September 21-24, 1992, A.A. Balkema, pp. 401-408.

Application of Stress-Wave Theory to Piles, F. B. J. Barends (ed.) © 1992 Balkema, Rotterdam. ISBN 90 5410 082 6

Dynamic and static testing for pile capacity in a fine-grained soil

B.H.Fellenius
University of Ottawa and Anna Geodynamics Inc., Ont., Canada

R.D.Edde & L.L.Beriault
Anna Geodynamics Inc., Ottawa, Ont., Canada

ABSTRACT Initial dynamic testing of seven, 35 m to 37 m long, 244 mm diameter, pipe piles at restriking between one and two weeks after EOID revealed variable and inadequate capacities and gave inconclusive evidence on soil set-up. An expanded testing programme supplemented with static testing indicated that the soil set-up took more than three weeks to develop fully. The programme also showed that the capacity determined in the static loading test agreed with capacity computed from the dynamic measurements; with CAPWAP as well as with the CMES methods RA2 and RMAX. The dynamic testing resolved the problem of choosing a termination criterion for the contract that combined depth with penetration resistance at EOID and at restriking.

INTRODUCTION

A piling project located near the city of Hull, Quebec, Canada, involved the installation of about 300 piles to support a waste recycling facility over an almost square area of 6,600 m^2. The piles were driven pipe piles distributed as single piles or in groups of two. The soil conditions were variable, that is, thickness of soil layers and depth to competent soil layers varied across the site. The groundwater table was close to the ground surface. Both dynamic and static tests were performed for quality control and inspection of the piling work to confirm termination criteria and pile capacities.

PILE, HAMMER, AND, SOIL INFORMATION

The piles were 244 mm diameter, closed-toe, driven, steel pipe piles of four different wall thickness and, therefore, different cross-sectional area. A design load of 750 KN was assigned to piles having a cross-sectional area of 60 cm^2 and 66 cm^2, and 1,100 KN (for a majority of the piles) to those with a cross-sectional area of 74 cm^2 and 81 cm^2. Two drop hammers were used to drive the piles, one with a 28 KN ram and one with a 30 KN ram. The piles were driven to embedment depths ranging from 34 m through 37 m (one pile was driven to 45 m). The assigned heights of fall ranged from 2.1 m through 3.6 m; most heights ranged from 3.0 m through 3.6 m.

The soils investigation at the site of the new building was limited in scope and consisted of nine boreholes spaced equidistantly about 35 m apart: four were terminated above a depth of 18 m; three were terminated at a depth of about 35 m, and two were advanced to about 45 m. The stratigraphy at the site was found to consist of three main strata: a clay fill of a thickness ranging from 8 m through 18 m underlain by an about 20 m thick layer of overconsolidated silty clay that gradually changed to clayey silt. The natural water content of the silty clay and clayey silt were about 60 % and 50 %, respectively, and the undrained shear strength, determined by means of a vane shear device, was about 100 KPa. The stress imposed by the building is smaller than the preconsolidation value of the silty clay and clayey silt which eliminates settlement concerns from the design. Dense glacial sand and clay till was found in three boreholes at depths varying from about 28 m through 40 m.

DYNAMIC TESTING

Selected piles were restruck with dynamic measurements by means of the Pile Driving Analyzer (PDA) according to procedures of the ASTM D4945 standard. The primary purpose of the PDA measurements was to determine the static capacity termed Case Method Estimate, CMES (Rausche et al., 1985). The J-factor dependent CMES value called RMAX was used for the project data together with a J-factor independent CMES value, called RA2.

CAPWAP analysis (Rausche et al., 1972) was performed on selected records from Beginning-of-Restrike, BOR, the first blow, to determine the pile capacity and calibrate the CMES capacities.

The design assumed that the piles were to be driven through the silty clay and clayey silt layer and derive most of the required bearing capacity in the glacial till. Based on prior experience in the area, it was expected that the piles could be terminated using a blow-count criterion. It was not recognized that penetration resistance (PRES) at end-of-initial-driving, EOID, in clay soils is not a reliable measure on final bearing capacity.

From the very start of the construction pile driving, a wide range of penetration resistance was observed at EOID: The penetration for one blow ranged from a few millimetres through as much as 150 mm. This was considered due to that the depth to the glacial till deposit differed across the site, that is, the length of pile embedment into the glacial till varied across the site. The number of boreholes was small for the size of the piling area and only three had been taken deep enough to determine the location of the glacial till. Therefore, the borehole records could not be used to match blow-count to embedment into the glacial till.

To investigate the bearing capacity, an initial testing programme was implemented consisting of dynamic measurements at restrike on seven piles (one day of testing) selected from across the site and performed 9 through 18 days after EOID, when all effects of the installation disturbance were thought to have abated. Six of the piles had been installed to a narrow range of embedment—from 34.4 m through 37.5 m—and one pile had been driven to a depth of 44.5 m. All piles had been driven in more or less the same way with no distinction made for whether the intended allowable load was 750 KN or 1,100 KN. The CMES values (calculated according to the RA2 method) indicated capacities ranging from 1,490 KN through 2,270 KN. The desired at-least capacity was twice the design load. That is, only one of the seven piles had a capacity that was adequate for the 1,100 KN load. Six of the seven piles had capacities adequate for the 750 KN load.

When analyzing the records, no consistency was found between time after EOID, embedment depth, and PRES values at restrike and EOID. A diagram of CMES capacity versus days after EOID is presented in Fig. 1. The diagram does suggest that the capacities increased with time after the initial driving, that is, presence of soil set-up is suggested. However, the indication is not clear. Nor can it be ascertained from the diagram whether or not the set-up trend would have continued beyond 18 days and toward values acceptable also for the 1,100 KN load.

A comparison is presented in Fig. 2 between the wave traces from Piles 125 and 25, which were restruck 9 days and 18 days after EOID,

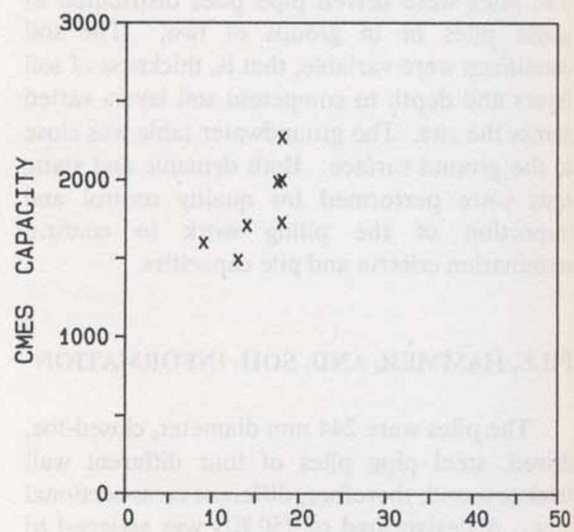

Fig. 1 CMES capacity versus days after EOID for the first seven piles

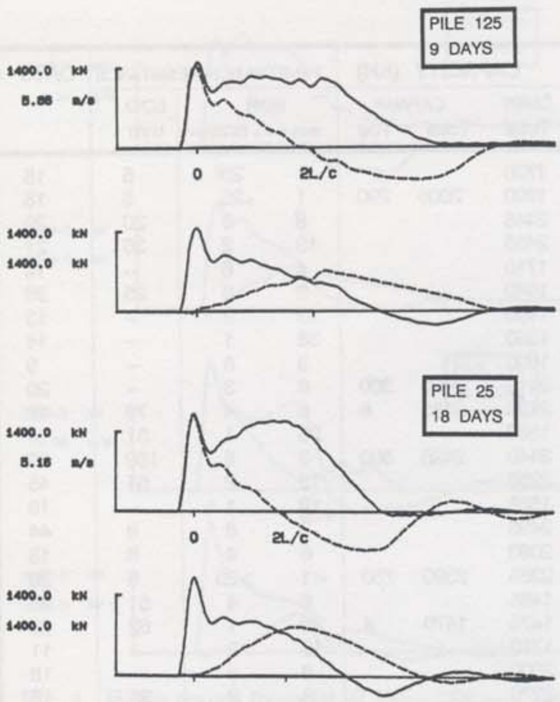

Fig. 2 Wave traces from BOR Piles 125 and 25
Solid lines: Force Wave, Wave-Down
Dashed lines: Velocity wave, Wave-Up

respectively. The wave traces are Force and Velocity and Wave-Up and Wave-Down. The two wave traces, as well as the five not shown here, suggest that the piles derive the bearing capacity mostly from shaft resistance. Furthermore, the traces confirm that the reason for the smaller capacity of Pile 125 (1,600 KN) as opposed to Pile 25 (1,990 KN) is that full set-up has not yet occurred at 9 days after EOID. Again, it is not possible to determine if full set-up has occurred for the longer wait (18 days). (For explanation on visual interpretation of wave traces, see Fellenius, 1984 and Hannigan, 1990).

Because the testing showed that six of the seven piles had inadequate capacity, it became central to the acceptance of the pile foundations to determine whether or not appreciable set-up continued to develop beyond 20 days after EOID. It was assumed that expanding the testing programme to include a larger number of piles across the site and testing them at restrike at different times after EOID would offset the uncertainty associated with the limited soil information.

Responding to the urgency of the project, 13 additional piles were restruck with dynamic measurements. Three piles were restruck a second time, seven days after the first occasion. To verify that the capacities determined in the dynamic test were comparable with static bearing capacity, the expanded testing programme included one static loading test to plunging failure.

The results, PDA and PRES data, of the measurements and analyses are compiled in TABLE I. The shaded pile numbers indicate the seven piles monitored initially. The CMES values indicated are the RA2 values. The RA2 method showed to provide more consistent agreement between CMES and CAPWAP which is not surprising considering the wide array of PRES values for the analyzed blows. As indicated by Fellenius and Riker (1988), the J-factor is not just a soil factor, it varies with pile size, time after EOID, and resistance distribution. For the piles tested at the subject site, the J-factors ranged from about 0.5 through 0.7. This range also agreed with the RMAX capacity values computed by CAPWAP and RA2.

The period between EOID and restriking ranged from 7 days through 44 days. Fig. 3 presents the CMES capacity versus Days after EOID for all piles monitored at first restrike.

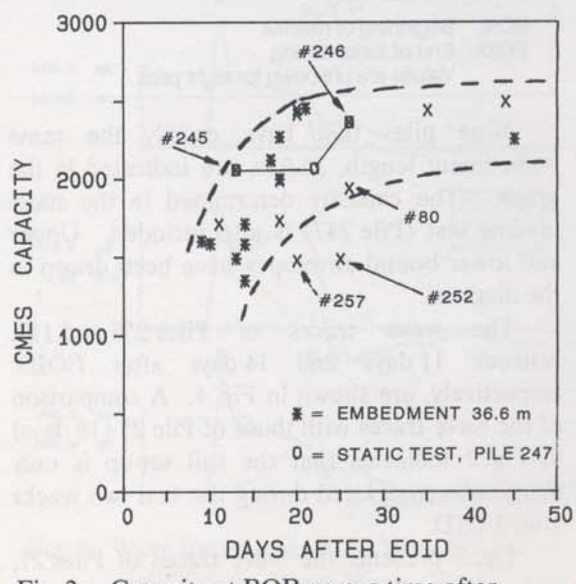

Fig. 3 Capacity at BOR versus time after EOID for all piles

TABLE I PDA and PRES DATA

PILE	DEPTH	LOAD	EMAX			FIMP	CAPACITY (KN)			PENETRATION RESISTANCE			DAYS
							CMES	CAPWAP		BOR		EOID	
#	m	KN	KJ	%	H (m)	KN	Total	Total	Toe	mm ==>	Bl/25mm	mm	
23	34.2	750	34	55	2.1	1350	1730			1	25	5	18
25	34.2	750	46	50	3.0	1610	1990	2005	290	1	25	5	18
27	35.7	750	66	60	3.7	1825	2445			8	3	20	20
55	36.6	750	73	65	3.7	1775	2455			13	2	30	21
76	36.6	750	46	65	2.5	1630	1710			4	6	–	14
80	34.4	1100	54	50	3.6	1875	1940			3	8	25	26
99	36.6	1100	39	55	2.4	1420	1490			13	2	–	13
119	36.6	1100	74	70	3.8	2295	1350			38	1	–	14
125	44.5	750	51	70	2.4	1485	1600			3	8	–	9
132	35.1	750	75	70	3.6	1920	2415	2400	300	8	3	–	20
141	36.6	1100	60	60	3.3	2135	2120	2185	6	6	4	76	17
144	36.6	1100	64	60	3.6	2080	1580			25	1	51	14
156	33.8	750	68	65	3.5	1820	2440	2425	600	3	8	150	35
188	36.6	1100	64	65	3.3	1995	2255			13	2	51	45
207	36.6	1100	58	55	3.5	2120	1585			19	1	–	10
213	33.5	1100	60	60	3.3	1975	2495			3	8	8	44
245	34.1	1100	62	55	3.6	2175	2060			6	4	8	13
246	33.5	1100	79	70	3.6	2480	2365	2390	780	<1	>25	6	26
252	35.1	1100	62	55	3.6	1950	1485			6	4	61	25
257	35.1	1100	65	60	3.6	2200	1475	1470	4	32	1	62	20
258	35.5	1100	64	60	3.6	1875	1710			13	2	–	11
266	37.5	750	58	65	3.0	1830	2000			8	3	–	18
272	36.6	750	68	75	3.0	1810	2270			8	3	25	18
Second Restrike													
25	43.2	750	48	45	3.5	1725	2025			3	8	5	25
99	36.6	1100	44	50	3.0	1340	1485			13	2	–	20
266	37.5	750	55	50	3.6	1800	1890			6	4	–	25
Static Loading Test													
247	33.5	1100	Failure load in the test = 2070 KN							–	–	8	22

LOAD: Design load
EMAX: Maximum transferred energy
% = Ratio to potential energy
FIMP: Impact force
BOR: Beginning of restrike
EOID: End of initial driving
Values are unknown for eight piles

H (m): Height-of-fall of drop hammer in metre at BOR
CMES: Case method estimate of static capacity
CAPWAP: Static capacity computed in CAPWAP
STATIC: Capacity determined in a static loading test
DAYS: Number of days between EOID and BOR

Nine piles that have exactly the same embedment length, 36.6 m, are indicated in the graph. The capacity determined in the static loading test (Pile 247) is also included. Upper and lower bound envelopes have been drawn in the diagram.

The wave traces of Piles 258 and 119, restruck 11 days and 14 days after EOID, respectively, are shown in Fig. 4. A comparison of the wave traces with those of Pile 25 (18 days) in Fig. 2 indicates that the soil set-up is only marginally completed during the first two weeks after EOID.

Fig. 5 presents the wave traces of Piles 27, and 156, restruck 20 days and 35 days after EOID. A comparison with the traces of Pile 25 in Fig. 2 indicates that the soil set-up continues beyond 20 days.

All wave traces suggest that all piles derive bearing capacity mostly from shaft resistance. The wave traces indicate that the longer the set-up time, the more capacity is gained in the lower portion of the piles, that is, in the clayey silt and glacial till.

Two pile records lie outside the upper and lower bound envelopes in Fig. 3: Piles 257 and 252, restruck 20 days and 25 days after EOID. Fig. 6 presents the wave traces of Piles 257 and 252. These are more similar to those from early restriking (shown in Fig. 5) than to those from later restriking (shown in Fig. 4). It would appear that Piles 252 and 257 have a

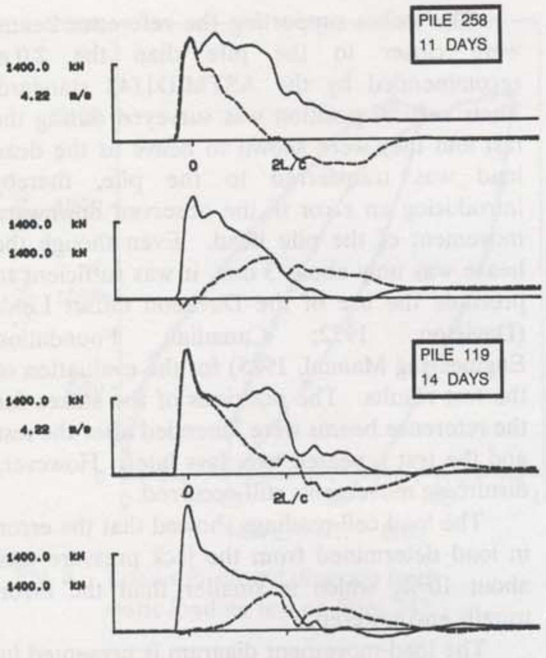

Fig. 4 Wave traces from BOR Piles 258 and 119
Solid lines: Force Wave, Wave-Down
Dashed lines: Velocity wave, Wave-Up

slower set-up development than the other piles tested. However, the traces indicate an absence of positive toe reflection. That is, the piles have practically no toe resistance. A CAPWAP analysis on Pile 257 computes a toe resistance of no more than 4 KN.

Table I contains both the total and the toe resistances computed in CAPWAP. Two of the piles show toe resistance values that exceed 600 KN and two show about 300 KN. The larger values are from piles tested beyond 25 days after EOID. In contrast, for two piles, Piles 257 (as mentioned) and Pile 141, the CAPWAP indicates essentially zero toe resistance despite the piles were tested as late as 17 days and 20 days after EOID. Common for Piles 257 and 141 is that the penetration per blow both at EOID and at BOR is large; at BOR the penetration corresponds to about 1 bl/25mm and 4 bl/25mm.

The values of penetration resistance at EOID and BOR, the shape of the wave traces, and the capacity values all point toward that Piles 252 and 257 have not been driven into the glacial till.

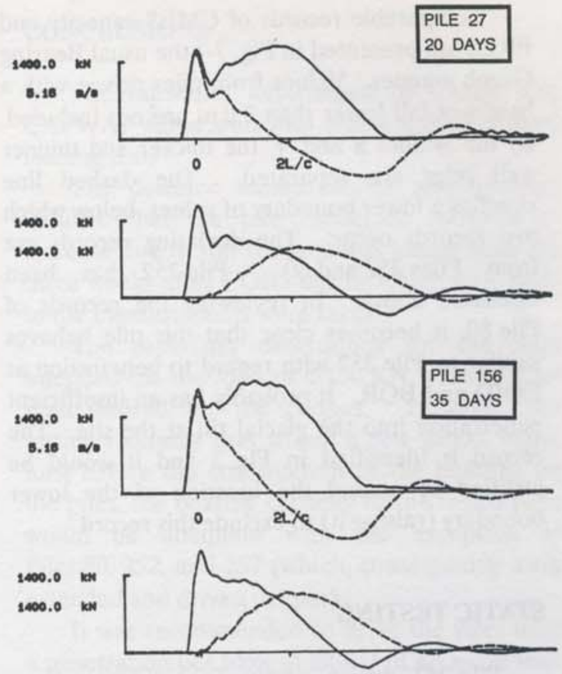

Fig. 5 Wave traces from BOR Piles 27 and 156
Solid lines: Force Wave, Wave-Down
Dashed lines: Velocity wave, Wave-Up

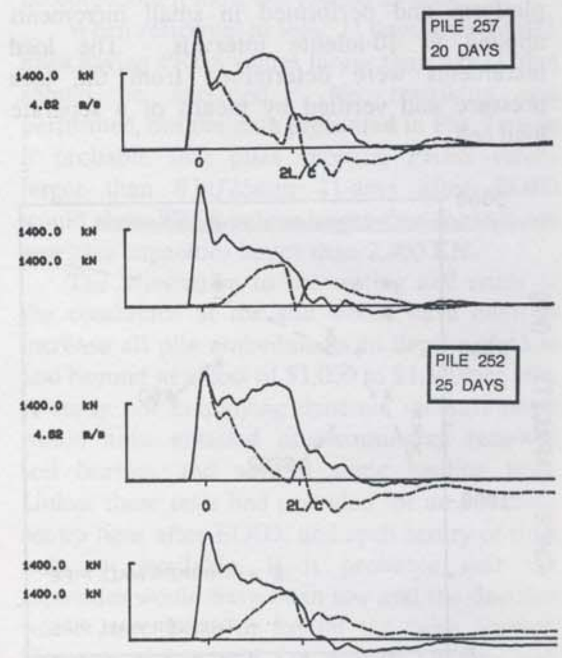

Fig. 6 Wave traces from BOR Piles 257 and 252
Solid lines: Force Wave, Wave-Down
Dashed lines: Velocity wave, Wave-Up

Comparable records of CMES capacity and PRES are presented in Fig. 7—the usual Bearing Graph manner. Values from piles driven with a height-of-fall lower than 3.0 m are not included. By the symbol **x** and **+** the thicker and thinner wall piles are separated. The dashed line signifies a lower boundary of values, below which two records occur. The deviating records are from Piles 252 and 80. Pile 252 has been discussed above. In reviewing the records of Pile 80, it becomes clear that this pile behaves similar to Pile 252 with regard to penetration at EOID and BOR. It probably has an insufficient penetration into the glacial till at the site. The record is identified in Fig. 3 and it would be justified to amend the location of the lower boundary (raising it) to exclude this record.

STATIC TESTING

Pile 247 driven within 5 m distance from Piles 245 and 246 and to almost the same embedment depth was subjected to a static loading test 22 days after EOID. The test, a quick test, was by jacking against a loaded platform and performed in small increments applied at 10-minute intervals. The load increments were determined from the jack pressure and verified by means of a separate load cell.

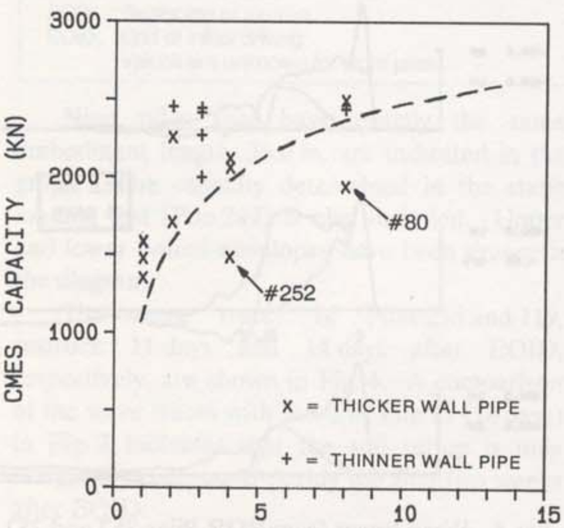

Fig. 7 CMES capacity versus PRES

The stakes supporting the reference beams were closer to the pile than the 2.0 m recommended by the ASTM D1143 standard. Their vertical position was surveyed during the test and they were shown to heave as the dead load was transferred to the pile, thereby introducing an error in the observed downward movement of the pile head. Even though the heave was only about 3 mm, it was sufficient to preclude the use of the Davisson Offset Limit (Davisson, 1972; Canadian Foundation Engineering Manual, 1985) for the evaluation of the test results. The positions of the stakes for the reference beams were amended after the test and the test repeated two days later. However, disturbing movements still occurred.

The load cell readings showed that the error in load determined from the jack pressure was about 10 %, which is smaller than the error usually encountered.

The load-movement diagram is presented in Fig. 8. The loads indicated are from the load cell readings. As the capacity of the pile is clearly reached in both tests, the uncertainty of the movement values do not matter. For the same reason, there is no need to determine the BrinchHansen Failure Load or the Chin Kondner Extrapolation Load (Canadian Foundation Engineering Manual, 1985). The peak load is 2,070 KN in the first test and 2,090 KN in the repeat test. It is interesting to note that the capacity of the repeat test is about the same as in the first test.

The data point of Pile 247 has been connected to those of Piles 245 and 246 in Fig. 3. Considering the slight difference in pile length and, more important, the difference in days after EOID and in penetration per blow at BOR, the agreement is excellent between the CMES and CAPWAP determined capacities and the failure load in the static loading test.

STATIC ANALYSIS OF CAPACITY

The load distribution for Pile 247, the pile for the static loading test, was calculated by means of the UNIPILE program (Goudreault and Fellenius, 1990) using both an effective stress (beta-method) and total stress (alpha-method). The density of the three soils layers

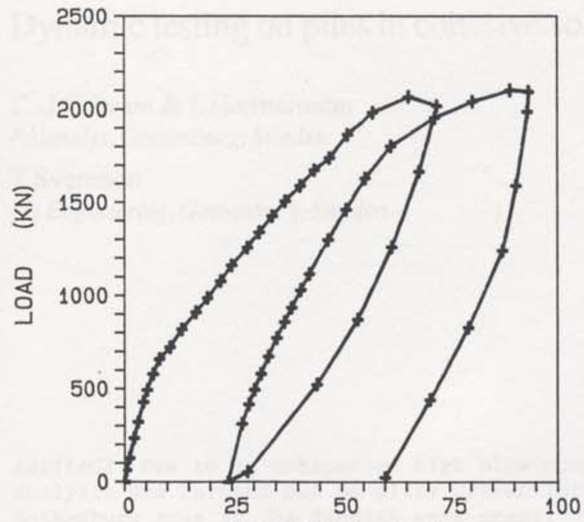

Fig. 8 Load-movement diagram from static loading test on Piles 247

were assumed to be 1,800 kg/m², 1,650 kg/m², and 2,100 kg/m², respectively. The pore pressures were assumed to be hydrostatically distributed from a groundwater table 0.5 m below grade. In the effective stress analysis, the beta-values assigned to the three layers were 0.3, 0.4, and 0.7. In the total stress analysis, the total shaft resistance in the clay fill was assumed to be 40 KN and a shaft shear of 100 KPa was assigned to the clay layers. In the till, effective stress was assumed and the earlier parameters were retained.

With the mentioned values, the capacity (2,070 KN) determined in the test was matched in the analyses by imposing a toe bearing coefficient, N_t, equal to 40 in the effective stress analysis. In the total stress analysis, matching the tested capacity required a mobilization of shaft shear of 67 KPa, which is about equal to two thirds of the vane shear strength, that is, alpha would be equal to 0.67.

The analyses do not claim to be correct. In fact, each single parameter is probably incorrect. However, when considered together, the ß and α values are conservative and this suggests that the test result is influenced by remaining excess pore pressures. It is, therefore, realistic to assume that the capacity of the test pile would have been larger had the time after the EOID been longer.

CONCLUSIONS

The capacities determined by CMES and CAPWAP agree well with that found in a static loading test.

The dynamic measurements and analyses indicate that the pile capacities continue to increase due to soil set-up much longer than the three weeks after EOID during which time most of the measurements were taken.

The capacities determined for the piles intended for the load of 1,100 KN were smaller than desired for the project. However, the testing indicates that within a few weeks more, long before the construction of the building on the piles, the bearing capacity of the tested piles would be adequate with the exception of Piles 80, 252, and 257 (which, consequently were extended and driven deeper).

It was recommended to drive the piles until a penetration per blow at EOID of no more than 8 mm/blow (that is, PRES to be larger than 3 bl/25mm) using a height-of-fall of 3.5 m. Beyond the embedment depth of 37 m, progressively larger penetration could be accepted, but it must not exceed 25 mm.

When restriking at least 21 days after EOID, piles having PRES values larger than 8 bl/25mm should be accepted. No restriking was performed, but the data presented in Fig. 7 make it probable that piles showing PRES values larger than 8 bl/25mm 21 days after EOID would ~~show PRES values larger than 8 bl/25mm and~~ have capacities larger than 2,400 KN.

The alternative to the testing and study of the conditions at the site would have been to increase all pile embedments to depths of 45 m and beyond at a cost of $1,000 to $1,500 per pile. A study not employing dynamic measurements would have entailed time-consuming renewed soil borings and several static loading tests. Unless these tests had provided for an adequate set-up time after EOID, and such luxury of time was not available, it is probable that the capacities would have been low and the decision would have been to extend the piles anyway. However, delays and the expense of time and money were avoided as the dynamic measurements gave the answer within a few days of testing and analysis while the construction proceeded without delay.

REFERENCES

CANADIAN FOUNDATION ENGINEERING MANUAL, 1985. Second Edition. Part 4: Excavations and Retaining Structures. Canadian Geotechnical Society, Technical Committee on Foundations, BiTech Publishers, Vancouver, 456 p.

DAVISSON, M. T., 1972. High capacity piles. Proceedings of Lecture Series on Innovations in Foundation Construction, American Society of Civil Engineers, ASCE, Illinois Section, Chicago, March 22, pp. 81 - 112.

FELLENIUS, B. H., 1984. Wave equation analysis and dynamic monitoring. Deep Foundations Journal, Deep Foundations Institute, Sparta, New Jersey, Vol. 1, No. 1, pp. 49 - 55

GOUDREAULT, P. A. and FELLENIUS, B. H., 1990. Unipile Version 1.0 Users Manual, Bengt Fellenius Consultants Inc., Ottawa, 76 p.

HANNIGAN, P. J., 1990. Dynamic monitoring and analysis of pile foundation installations. Deep Foundations Institute, Sparta, New Jersey, 69 p.

RAUSCHE, F., GOBLE. G. G., and LIKINS, G. E., 1985. Dynamic determination of pile capacity. American Society of Civil Engineers, ASCE, Journal of Geotechnical Engineering, Vol. 111, GT3, pp. 367383.

RAUSCHE, F., MOSES, F., and GOBLE, G. G., 1972. Soil resistance predictions from pile dynamics. American Society of Civil Engineers, ASCE, Journal of Soil Mechanics and Foundation Engineering, Vol. 98, SM9, pp. 917–937.

RIKER, R. E. and FELLENIUS, B. H., 1988. Case method estimate for piles in glacial soils. Proceedings of the Third International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, May 25 - 27, 1988, pp. 565 - 578.