Load Capacity of Pipe Piles in Cohesive Ground

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Abstract

Very often highway departments drive test piles at proposed bridge locations to refusal or to a predetermined set and utilize these records to determine the pile capacity using the Hiley and Engineering News Record pile driving formulae This test pile driving also provides information on the depth to which the piles can be driven and on problems that may likely be encountered during production piling The pile capacity obtained from pile driving formulae is generally used by the structural engineer to undertake the preliminary design of the bridge foundations The use of the pile driving approach to capacity determination often works well when the ground is competent at relatively shallow depths However, where piles cannot achieve refusal unless driven into stiff or hard ground a geotechnical evaluation of pile capacity becomes more relevant and is often relied upon for pile capacity determination This paper describes a site where H-pile and closed end pipe piles attained refusal at a depth of 31 metres in hard clay till and where the geotechnical evaluation recommended that the pier piles be terminated at a higher elevation To demonstrate that the geotechnical recommendations were acceptable, static load testing and Pile Driving Analyzer tests were undertaken The detailed testing program demonstrated that the driving of piles to refusal was not necessary to achieve the desired pile capacities and that conventional static analysis provided capacities that were sufficiently reliable for design

Introduction

As part of the upgrading of various bridge structures in the Province of Alberta, Alberta Transportation (Provincial Department of Transportation of the Government of Alberta) initiated in 1988 the design and construction of a new bridge at an existing bridge site across the Paddle River This site is located along a local road at approximately 175 km North West of Edmonton, the Provincial Capital of Alberta, and 16 km south of the Village of Greencourt. The site location is shown on the vicinity map in Figure 1.

The proposed bridge was a two-span steel girder structure 46 m in length founded on pipe pile piers and H pile abutments Each pier was to consist of five (5) pipe piles

Preliminary Geotechnical Investigation and Test Pile Driving

The preliminary geotechnical investigation was undertaken by the Geotechnical Services Section of the Department in March/April 1988 by drilling two test holes Each hole was

located within the existing shoulder area on either side of the existing bridge and not far from the respective bridge abutments Figure 2 shows a typical cross section of the bridge site location of the test holes and typical soil logs Historical information from the files of the existing bridge indicates that a timber test pile reached refusal at an elevation of 715 5 m, approximately eight (8) m below the elevation of the river bed

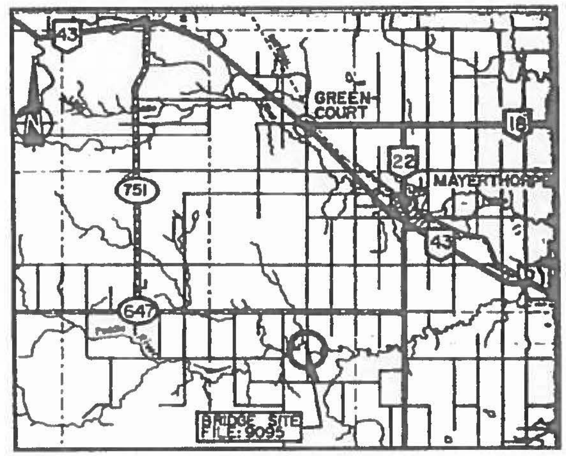

Figure 1. Vicinity Map

Test hole #1 was drilled to a depth of 26 4 m while TH #2 was drilled to 35 5 meters The soil profile consisted of approximately 4 5 m of silty clay overlying about 1.0 m of sand Below the sand a silty sandy gravelly clay till stratum was encountered to the end of the depth of drilling in each testhole In general, the till possessed intermediate plasticity with a mean LL of 43%, PL of 18%, and an average moisture content of 24%.

The consistency of the till varied with depth Standard Penetration Test (SPT) blow counts in the till above elevation 713 m ranged from 7 to 17 blows with a mean of 13 blows Below that elevation, the blow counts increased to a range of 19 to 38 blows with a mean of 25 blows

Based on an evaluation of the drilling information, a geotechnical report (Diyaljee and Umadat, 1989) was submitted, in which preliminary recommendations were made regarding the type of foundations for the abutments and the piers For the abutments 310 mm x 94 kg steel H piles driven to a tip elevation of 712 0 m or pipe piles of 508 mm diameter driven to an elevation of 714.0 m were recommended Based on total stress analysis method, the corresponding allowable pile capacities were estimated to be 550 kN (55 tonnes) for an H- pile and 700 kN (70 tonnes) for a pipe pile A maximum settlement of 3 mm was projected for each of the pile.

For the piers, closed end steel pipe piles driven to a tip elevation of 711 0 m were recommended The corresponding allowable pile capacity for a typical 610 mm diameter pipe pile was estimated to be 1000 kN for a Factor of safety of 3 The corresponding settlement was about 5 mm In comparison, the allowable pile capacity for this pile using the Pile Driver's Guide (Peterson, 1977) used by the Department to determine allowable design capacities of piles without load-settlement testing was determined to be 1062 kN for a factor of safety of 3 for a pile achieving practical refusal i e in this case a tip elevation of 693 5 m This allowable capacity was determined by multiplying the outside diameter in mm by 1 75 kN This is an empirical relationship, the origin of which is unknown but appears to be obtained from experience with the use of the pile driving formulae

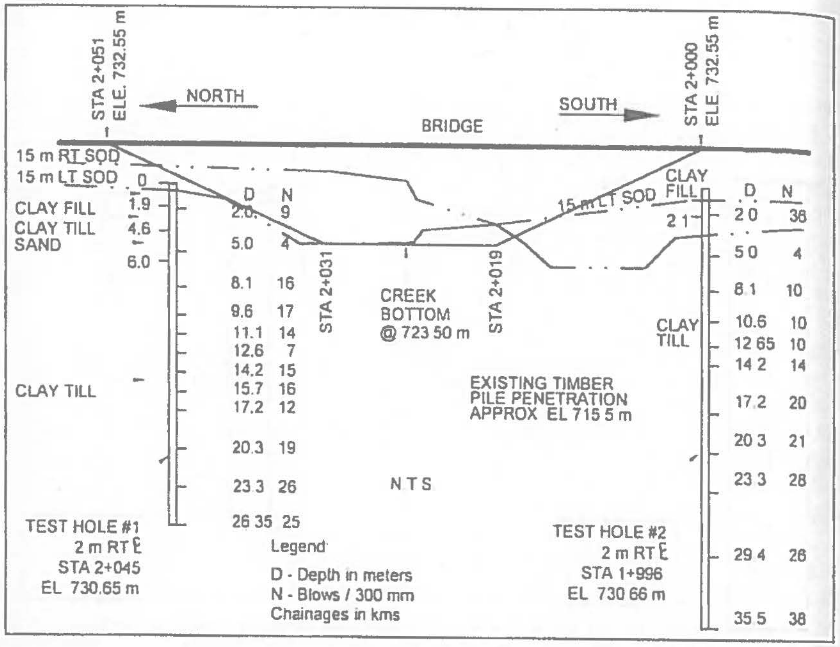

Figure 2. Typical Cross Section of Bridge Site

Almost concurrent with the timing of the preliminary geotechnical investigation, driving of two test piles (a 310 mm x 94 kg H pile and a closed end pipe pile of 355 mm diameter) was undertaken by the Bridge Engineering Branch in March 1989 These test piles were driven on the north side of the river in the vicinity of the existing bridge using a Hera 1500 single acting diesel hammer with a rated maximum energy of 40 6 kJ

The location of the pipe pile was chosen close to the bottom of the creek while the H pile was driven further up the north bank. The cross sectional shapes of the two piles were selected different from each other because of the common practice of using H piles for abutment and pipe piles for piers.

The test piles driving results (Figure 3) indicated that refusal was achieved at an embedment depth of 31 meters for the two piles, approximately at elevation 699 8 m for the H pile and elevation 693 5 m for the pipe pile

Subsequent Discussions Leading to Further Testing

Following the submission of the preliminary geotechnical report, no serious discussions took place until the preliminary design of the bridge was undertaken in May 1990 At that point, the consultation process increased between the Bridge Engineering Branch and the Geotechnical Services Section concerning the recommended depth of pile embedment During the discussions that followed, a debate arose regarding whether the abutment and the pier piles should be driven to refusal as done conventionally at most of the bridge sites or should be stopped at higher elevations as recommended The Bridge Engineering Branch was also concerned about terminating the pile tips in a zone of low blow counts indicated by the test pile driving

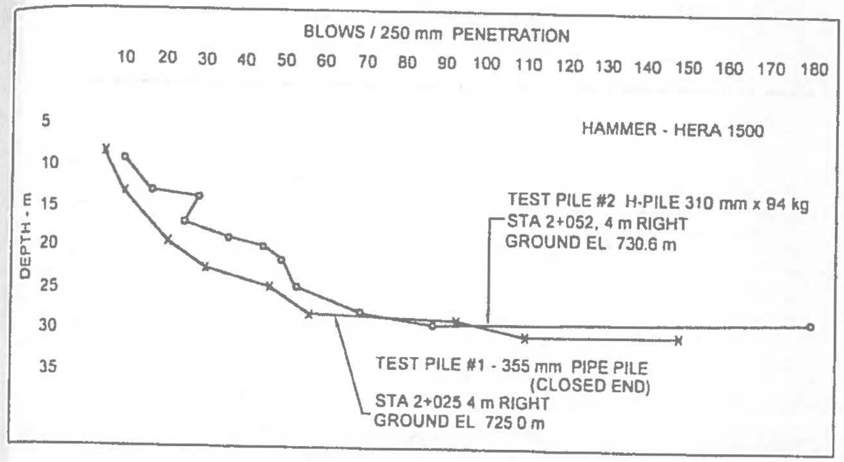

Figure 3. Test Piles Driving Record ***(March 1989)***

A re-evaluation of the drilling information at the identified locations of the abutments and piers was undertaken and a recommendation was made to drive the pipe piles of the piers to the elevation of 707 0 m (4 metres lower than 711 0 m originally recommended) Based on revised calculations using total stress approach, allowable pile capacities of 871 kN and 1008 kN were derived for a 610 mm diameter pipe pile corresponding to a factor of safety of 2 5 and tip elevation of 711 0 m and 707 m, respectively The larger capacity for the pile provided in the 1989 preliminary report recommendation resulted from a longer length of pile being analyzed as the actual location of the piers was unknown at the time Accounting for a decreased shaft length the comparable allowable capacity would have been capacity 857 kN for a factor of safety used initially

It was also suggested at the same time that further field testing would be advisable in terms of a static load test to determine the actual capacity of a production pipe pile driven to the two different elevations 711 0 m and 707 0 m The principle of static load testing was accepted and the scope of testing was also further enlarged to include Pile Driving Analyzer (PDA) testing

It should be noted that undertaking these tests were not the norm for the Department since only a single static load test was known to have been conducted in 1963 while PDA testing was used periodically by the Geotechnical Section since 1986 to substantiate geotechnical capacities determined from static analysis and to influence the Bridge Engineering Branch on the benefits of utilizing this form of testing during their test pile driving

The proposed testing program gave rise to a small field research project aimed at demonstrating to the structural engineers of the Department that adequate capacities could be obtained without driving piles to refusal and to increase the level of confidence of the geotechnical engineers on their design approach in providing geotechnical pile capacities using static analysis If this was proven, then the concept would indirectly reduce pile foundation costs in the long run for similar site conditions

Briefly, the schematics of the new testing involved (a) driving two closed end pipe piles to different elevations and undertaking the PDA testing as the pile tip moved downwards, (b) re-striking the piles at the end of a two-week setup period and repeating the PDA testing, and (c) subjecting the piles to a static load test as the last phase of the testing program

The opportunity was also taken to undertake cone penetrometer testing of the subsoil stratigraphy, and to install piezometers in the ground around the test pile locations

Details of Testing

Site Preparation and Installation of Piezometers. The various field related tasks of the pile testing program were undertaken between October 1, 1990 and November 15, 1990 Very close coordination of different activities was maintained between different sections of the Department Site access was prepared first and the locations of test piles were marked in the field in a relatively flat area on the west side of the local road situated on the north side of Paddle River

The installation of piezometers was carried out between October 12 and 15, 1990 Two test holes were drilled near the proposed location of the test piles to a depth of about 20 and 24 meters using an auger rig. Three (3) high air-entry piezometer tips were installed at different depths in each of the two holes (Newman and Weins, 1990) These piezometers were monitored during the pile driving operations as well as at the time of the PDA and static load testing

Because of sloughing of the wet sand at the bottom of the holes and a high water table, added care was taken to keep the holes open until the tips were installed and filter material placed around the tips.

Test pile Driving, PDA Testing and Static Load Testing. The pile testing program consisted of driving two single piles with dynamic monitoring carried out at the end of the initial driving by means of the Pile Driving Analyzer, PDA The piles were 324 mm (12 75 inch) diameter steel pipe piles having a wall thickness of 11 mm (0.44 inch) and a cross sectional area of 110 cm^2 (17 in^2) placed on a relatively flat ground and about 4 m apart Driving of three anchor piles was also undertaken for the subsequent static load testing.

Although it was identified to use 610 mm diameter production piles for the piers, 324 mm diameter piles were selected for static load and PDA testing, because of their ready availability at the time and overall less expenditure for static load testing in comparison with the use of 610 mm piles for such testing

The two test piles (called Pile 1- longer pile and Pile 2 - shorter pile) were driven to different embedment depths, viz , Pile 1 to 20 25 m depth and Pile 2 to 16.25 m depth Different depths were selected for the two piles to study the variation in response of the subsoil at the depths where the production pile tips would likely be located. A photograph of the test piles and anchor piles is shown in Figure 4

Driving of the test piles and the anchor piles was done between October 16 and 17, 1990 The two test piles, Piles 1 and 2, were first driven on October 16, to depths of 19.25 m (Pile 1) and 15.25 m (Pile 2) The next day, October 17, 1990, the initial driving was resumed with dynamic monitoring for an additional penetration of about 1.0 m taking Pile 1 to an embedment depth of 20 25 m and Pile 2 to a depth of 16.25 m. The piles were restruck with PDA measurements on October 31, 1990 for an additional penetration of 50 mm

The initial driving on October 16 and 17 was undertaken using a Hera 1500, single acting diesel hammer Restriking on October 31 was undertaken using an 18 kN (4,000 lb) drop hammer with heights of fall of 1.8 m and 2 4 m for Piles 1 and 2, respectively.

Following the completion of the re-striking test, static loading tests were conducted to failure on both the piles on two separate occasions - two weeks and four weeks after the initial driving (November 1, the day after the re-striking and November 14, 1990) using ASTM D-1143 quick maintained load procedure

Following the completion of the PDA and static load tests, an overall review of the testing information was undertaken and revised ultimate and allowable pile capacities were provided for the 610 mm pipe piles of the piers corresponding to pile tip elevations of 711.0 m and 707 0 m (Diyaljee, 1990)

The test pile driving was organized through the Bridge Engineering Branch of the Department, while the PDA tests were done by Anna Geodynamics Ltd of Ottawa, Canada Typical photographs of the PDA testing and static load test are shown in Figures 5 through 7

Cone Penetration Testing. This testing was undertaken through Conetec Inc. of Vancouver on October 23, 1990 The necessary drilling equipment was provided by Mobile Augers Research Ltd The testing was done in one hole to a depth of 20 meters along the river bank within the vicinity of the test piles Measurements were taken for the end bearing, sleeve friction and pore pressure at 0 25 m depth intervals as the cone was pushed into the ground (Mobile Augers and Research Ltd , 1990) The results of the cone penetrometer test are shown in Figure 8

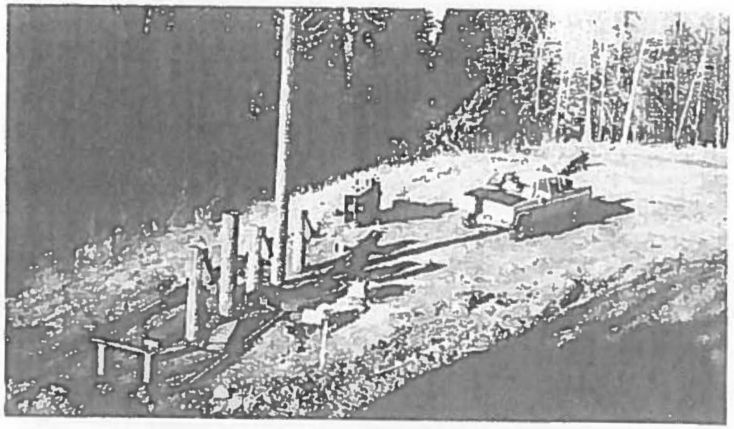

Figure 4. Setup of Test Piles and Anchor Piles

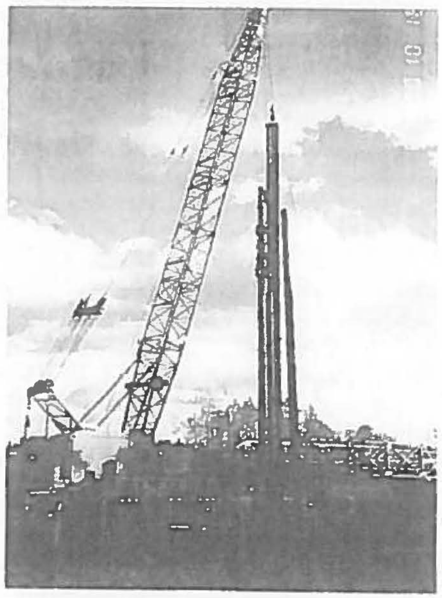

Figure 5. Pile Driving Setup

Figure 6. Restrike of Pile with Drop Hammer during PDA Testing

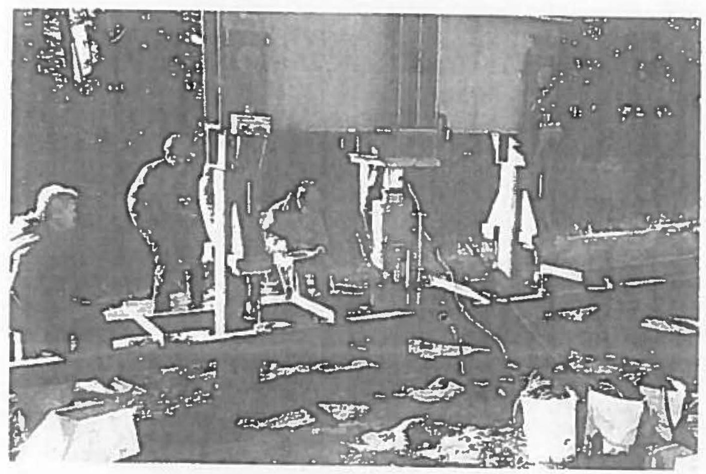

Figure 7. Static Pile Load Test in Progress

Analysis of Results

Cone Penetrometer Testing. From the observed readings of the cone penetrometer testing shown in Figure 8, the subsurface soil stratigraphy was identified to be generally homogeneous soft silty sand to silty clay but with denser seams at depths of about 8 m and 16 m. However, at about 22 m, the sleeve friction shows a significant drop in value that does not have a corresponding change in the point resistance

The pore pressures generated by the cone decreased significantly between depths 16 m and 18 m suggesting that the soil in this zone is coarser than above and below the zone Below 22 m, on the other hand, an increase in pore pressure was noted suggesting that the soil is finer in this zone than above this depth

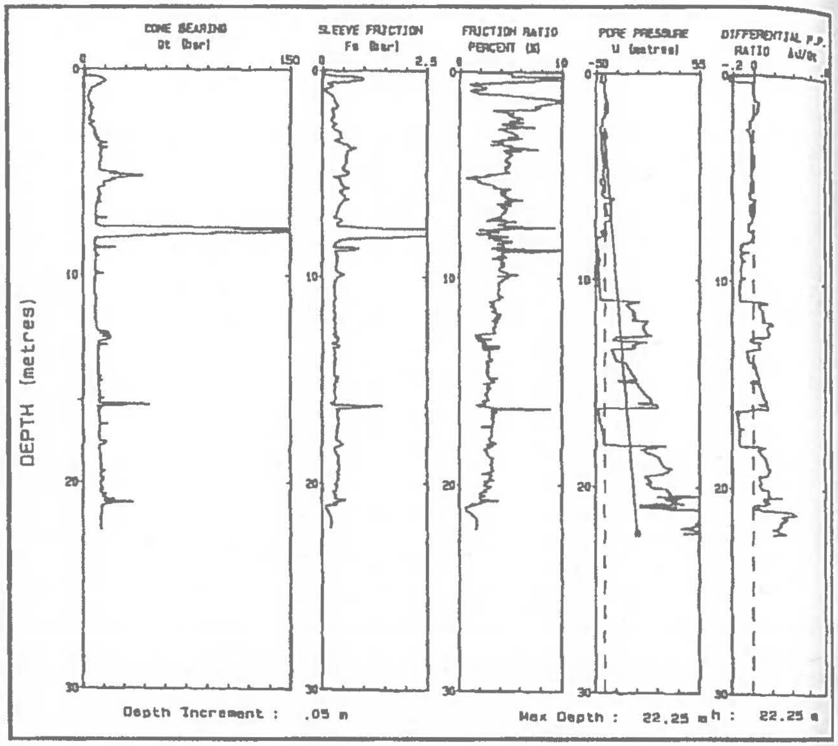

Figure 8. Cone Penetrometer Testing Results.

Piezometers Monitoring. The piezometers that were installed close to the two test piles were monitored immediately before the pile driving (for initial readings) and subsequently through the pile driving, re-striking and the static load testing. Figure 9 shows a typical set of piezometer readings taken in a hole at different times during the entire testing program.

The measurements showed that although excess pore pressures were generated during the pile driving, they had dissipated considerably by the time the static load testing was done The dissipation was faster in the top 9 m zone of the soil. Below 9 m, some excess pore

pressure still existed during the first static tests Some of this pressure, but not all, had dissipated by the time the second tests were carried out

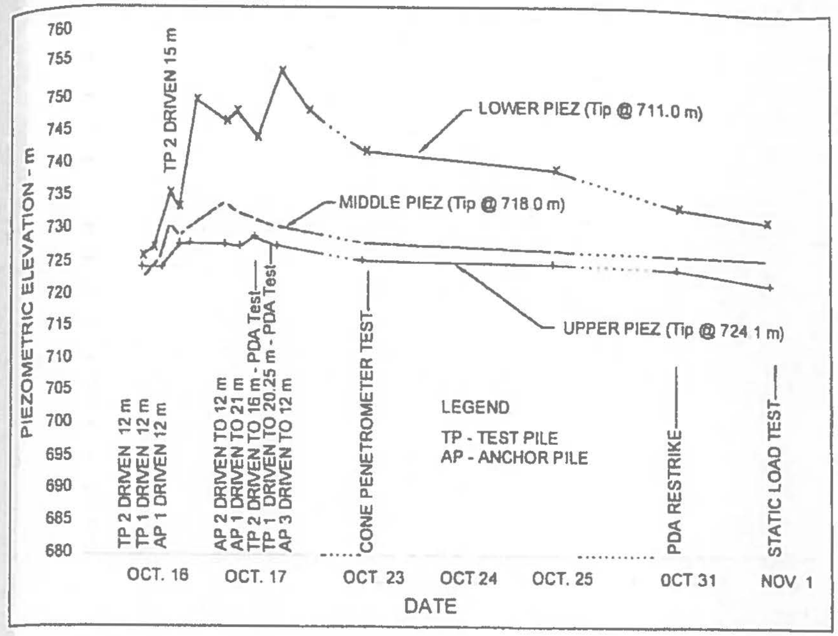

Figure 9. Typical Piezometer Plots

PDA Testing Results. (Fellenius, 1991-A) The end-of-initial driving (EOID) dynamic measurements of October 17, 1990 indicated that the energy transferred by the Hera 1500 diesel hammer ranged from about 8 kJ through about 10 kJ with a corresponding ratio of transferred energy to nominal energy of the hammer of about 20%-25%. The penetration resistance (PRES) for the last few blows for both piles were about 4 blows/25 mm The maximum force occurred at impact ranged from 1,220 kN (122 tonnes) through 1,500 kN (150 tonnes) corresponding to maximum stresses of 110 MPa (1,025 t/ft^2) through 135 MPa (1,258 t/ft^2)

The maximum activated static resistance at EOID as evaluated by the CMES-RMX method using a J-factor of 0 4 were 610 kN and 470 kN for Piles 1 and 2 respectively A Case Pile Wave Analysis Program (CAPWAP) (Rausche et al, 1972) analysis performed on both piles indicated an activated static bearing capacity of 535 kN (53 5 tonnes) and 470 kN (47 tonnes) at EOID for Piles 1 and 2, respectively

The dynamic measurements at restrike on October 31, 1990, indicated that the energy transferred by the 18 kN (4,000 lb) drop hammer using heights-of-fall of 1 8 m and 2 4 m ranged from about 20 kJ through to about 22 kN and from about 24 kJ through to about 28 kJ respectively, corresponding to ratios of transferred energy to nominal energy of the hammer of about 55%-65% Five blows were applied to Pile 1 and four blows to Pile 2 causing both piles to penetrate about 50 mm The penetrations correspond to an average equivalent penetration resistance PRES for both piles of about 2 blows/25 mm. The maximum force occurred at impact and ranged from 1,820 kN (182 tonnes) through 2,180 kN (218 tonnes) corresponding to maximum stresses of 165 MPa (1,538 t/ft^2) through 200 MPa (1,864 t/ft^2)

The maximum activated static resistances, as evaluated for the first Restrike (RSTR) blow by the CMES-RMX method using a J-factor of 0 4 were 1585 kN and 1135 kN for Piles 1 and 2 respectively The last restrike blow indicated maximum CMES-RMX resistances of 1255 kN and 850 kN respectively for the two piles

CAPWAP analysis indicated an activated static bearing capacity of 1505 kN and 1150 kN at Beginning-of-Restrike (BOR) for Piles 1 and 2 respectively. At End-Of-Restrike (EOR), the CAPWAP capacities were 1225 kN and 865 kN respectively. The reduction of capacity between the BOR and EOR values are considered to be associated with excess pore pressure being induced and accumulated for each blow.

It was assumed at the time of the restrike monitoring that all disturbances from the pile installation had dissipated including the excess pore pressures induced by the pile driving. Then, the CAPWAP capacity determined for the BOR records would be representative for the pile capacity while the CAPWAP capacity determined for the EOR records would be representative for the capacities at the time of the static testing conducted the following day

Static load tests. (Fellenius, 1991-B) The load movement plots for the two piles corresponding to the two static load tests are shown in Figures 10 and 11 respectively The typical nature noticeable of all the four static load tests was the occurrence of sudden failure Also, both piles show an increased capacity between the first and the second test, 20% for the Pile 1, the longer pile and 10% for Pile 2, the short one

For Pile 1 (the longer pile), the failure loads corresponding to the two static load tests are 816 kN (82 tonnes) and 979 kN (98 tonnes) respectively For Pile 2 (the shorter pile) similar failure loads observed are 734 kN (74 tonnes) and 801 kN (80 tonnes) respectively That the longer pile demonstrates a larger increase suggests that the increase occurred in the lower portions of the soil profile, that is in the zone of continued pore pressure dissipation (and increase of effective stress).

The static load testing results were also analyzed using three different interpretation approaches (Canadian Foundation Engineering Manual - 2nd Edition) and are presented in Table 1 The three approaches used respectively are (a) the Davisson Offset Limit Load method, (b) the Brinch-Hansen failure criterion, (c) the Chin Extrapolation criterion Table 1 also shows pile capacity values estimated from UNIPILE Program and effective stress approach (Fellenius, 1990) and using a value of N_t=30 for the toe bearing coefficient and a Beta Coefficient of 0 6 to 0 8 for different layers of the subsurface soil matrix. It is interesting to note that the ultimate pile capacity values derived by the different approaches for each pile are in reasonable agreement

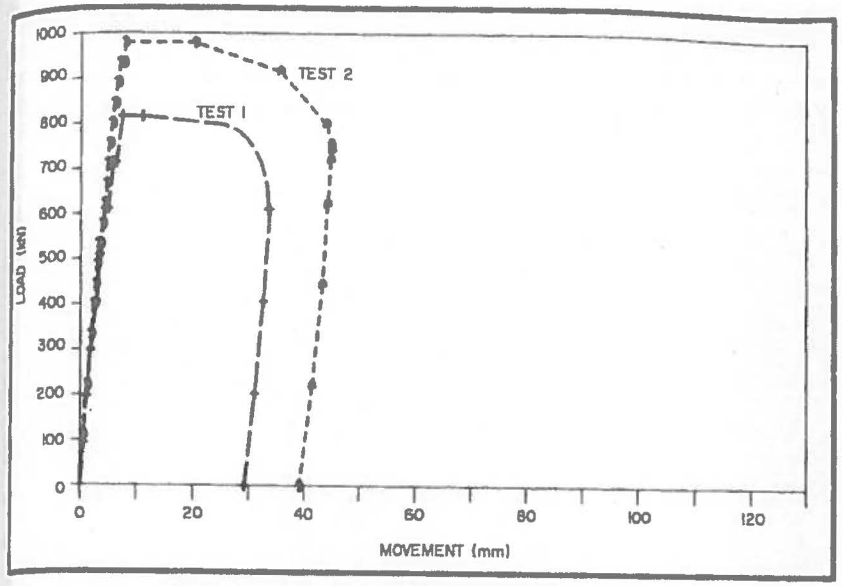

Figure 10. Typical Load Movement Plots for Pile 1

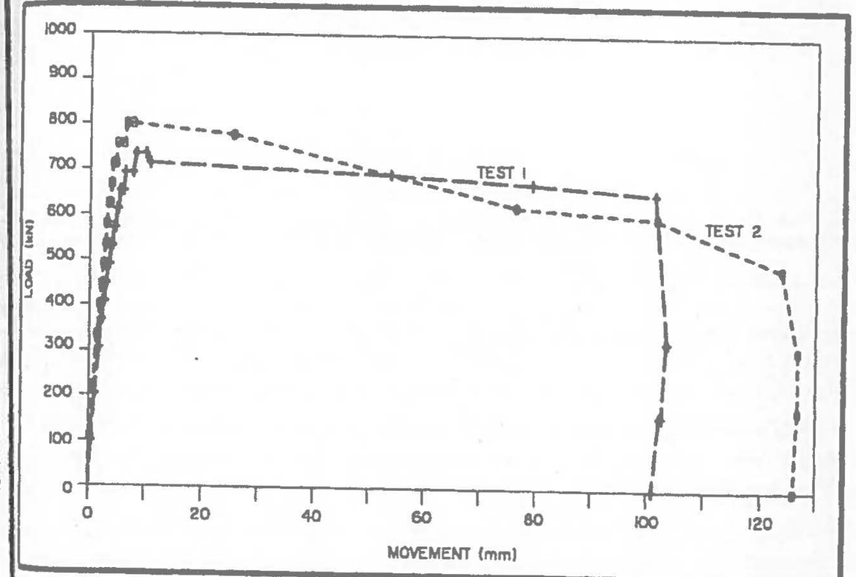

Figure 11. Typical Load Movement Plots for Pile 2

Table 1.
Comparison of Results of Pile Tests for 324 mm Diameter Test Piles

Details		Pile 1	Pile 2
Ground Elevation		727 4 m	727 3 m
Pile tip Elevation		707 0 m	711 m
Embedment Depth		20 4 m	16 3 m
Static Pile Load Test Results	Nov 1/90 *(Test 1)*	816 kN	734 kN
	Nov 14/90 *(Test 2)*	979 kN	801 kN
Interpretation of Static Pile Load Test results	Brinch-Hansen	1,000 kN	860 kN
	Chin Extrapolation	1,100 kN	1,000 kN
	Davisson Offset Limit	920 kN	830 kN
	Average of the above	1,007 kN	897 kN
Static Resistance Calculations *for Test 1* by effective stress method and UNIPILE Program	Shaft Resistance	1,032 kN	701 kN
	Toe Resistance	314 kN	156 kN
	Total	1,346 kN	857 kN
PDA Test Results	Oct 17/90 (Using Diesel Hammer)	535 kN	475 kN
	Oct 31/90 -(Using Drop Hammer)	1,220 kN	870 kN

Capacity of 610 mm Production Piles

Since the piers were to be constructed with 610 mm pipe piles, extrapolation of the ultimate pile capacities obtained from the pile load tests of the 324 mm piles was done to determine the capacities of the 610 mm size piles (Diyaljee and Cheng, 1990). Pile capacities calculated by different methods for the two test piles and the production piles are shown in Table 2

For the extrapolation, the observed pile capacities of the 324 mm piles were first subdivided into two components viz , adhesion (shaft resistance) and end bearing (toe resistance) capacities based on a ratio of shaft resistance/toe resistance derived from the effective stress approach These component capacities of the 324 mm test piles were then modified proportionally according to the ratios of 610 mm and 324 mm pile circumferences, pile toe areas and pile depths These results are presented in Table 2

The ultimate capacities predicted thus for 610 mm piles were 2,021 kN (202 tonnes) and 1,539 kN (154 tonnes) for pile tip elevations of 707 m and 711 m respectively Applying a factor of safety of 2, allowable capacities of 1,010 kN (100 tonnes) and 770 kN (77 tonnes) were recommended for the design It is interesting to note that similar allowable pile capacities were arrived at by applying a factor of safety of 2 5 to the revised ultimate pile capacities projected by the Geotechnical Services Section in their revised report of June 8, 1990. The higher factor of safety was chosen since no field testing was done at that time.

The load capacities projected for the 610 mm production piles based on the PDA test results of the 324 mm piles are also shown in Table 2 Using the Unipile program (Goudreault and Fellenius, 1990), Fellenius calculated the ultimate pile capacities of the 610 mm production piles corresponding to the two pile elevations These were determined to be 2780 kN and 1646 kN corresponding to the pile tip elevations of 707 m and 711 m respectively and are also shown in Table 2

Inference and Discussion

In general, the allowable pile capacities derived by different methods as shown in Table 2 for the 610 mm dia production pipe pile with the tip at 707 m elevation are in close agreement with each other However, for the pile with tip at 711 0 m elevation, there is some scatter in the pile capacity values varying from 650 kN to 870 kN This exercise also proved that it is not necessary to take the driven piles to a hard bottom, provided due diligence is exercised in making interpretations of the geotechnical information of each project . This exercise allowed the use of 17 m long piles instead of 31 m long piles to the hard bottom, thus indirectly saving some costs The results confirmed as well the approach used by the Geotechnical Section in providing pile capacities using the static method of analysis It should be pointed out that in this approach the choice of parametric values for the determination of skin friction and base resistance is based on experience and do not necessarily reflect actual values obtained from laboratory or field testing To-date, the bridge structure is performing well

Conclusion

The following conclusions can be derived from the findings of this case study

- Desirable allowable pile capacities for design can be obtained without driving piles to practical refusal This would reduce foundation costs in ground where refusal can only be achieved at a great depth

- For the 610 mm pipe piles proposed to be driven to elevation 711 0 m, allowable pile capacities derived from conventional static analysis using total stress and effective stress analyses and the Pile Driving Analyzer (PDA) test results gave values similar to that projected from the pile load testing

- For the 610 mm diameter piles proposed to be driven to elevation 707 0 m, allowable pile capacities derived from conventional static analysis using the total stress and effective stress approaches, and the PDA test results gave values that were about 100 kN larger for the total stress derived capacity and 100 kN lower for the effective stress derived capacity and PDA testing than the capacity projected from the pile load testing

Table 2.

Predicted Ultimate Load Capacities for 610 mm Pipe Piles (Based on Pile Load Test Results of 324 mm Diameter Pipe Piles)

Details	Pile 1- Long Pile *(324 mm dia)*	Pile 2 - Short Pile *(324 mm dia)*	For Production piles (610 mm dia)	
Ground Elevation (m)	727.4 m	727.3 m	724.5 m	724.5 m
Pile tip elevation	707.0 m	711.0 m	707.0 m	711.0 m
Embedment Depth	20.4 m	16.3 m	17.5 m	13.5 m
Load Capacity suggested in Geotechnical Section's revised Report of June 1990			2517 kN - Ultimate. 1006 kN - allowable for a F.S of 2.5	2178 kN - Ultimate. 871 kN - allowable for a F.S of 2.5
Ultimate Capacity and corresponding components predicted from Static Load Testing (of Nov 14, 1990) (See also Table 1)	979 kN- Ultimate = Shaft 751 kN + Toe 228 kN	801 kN - Ultimate = Shaft 655 kN + Toe 146 kN		
Pile Load Capacities of Production Piles Extrapolated from Static Load Tests	Ultimate		Shaft- 1313 kN Toe - 808 kN Total 2021 kN	Shaft- 1021 kN Toe- 518 kN Total 1539 kN
	Allowable (For a Factor of Safety of 2.0)		1010 kN	770 kN
Projected Load Capacity based on PDA test results			2520 kN- Ultimate 1008 kN - allowable for a F.S of 2.5	1646 kN - Ultimate. 668 kN - allowable for a F.S of 2.5
Load Capacity Projected by Fellenius based on UNIPILE program and effective stress approach			2780 kN - Ultimate. 1112 kN - allowable for a F.S of 2.5	1646 kN - Ultimate. 658 kN - allowable for a F.S of 2.5

at the corresponding tip elevation Such variation in results is generally expected in practice and can be attributed to a variety of reasons.

- Determination of allowable pile capacities using conventional static analysis using the total stress approach can be relied upon for design purposes.

- The use of Pile Driving Analyzer testing in test pile driving will allow more realistic pile capacities to be determined with depth and hence provide results which can be used with confidence to correlate with the capacity values derived from conventional static analysis

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