Influence of Subsoil Characteristics on Embedment Depths and Load Capacity of Large Diameter Pipe Piles

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Abstract

Driven H-piles and pipe piles are frequently used as substructure units for highway bri constructed in the Province of Alberta, Canada While geotechnical investigations a recommendations often precede the design of the substructure units, test piles are traditio driven at sites to determine the capacity of the piles using "so called" pile driving formul the efficiency of installation of the driven piles and to assess what pile lengths are requir Where geotechnical investigation and pile driving information correlate, there is gener confidence in the proposed piling scheme Where there are differences, very often the pi driving results tend to govern except in circumstances whereby the results show exceedin large variances in pile lengths This was the situation at a bridge site across Battle River i the Province of Alberta Pile driving records of this site showed piles reaching refusal at depth of about 55 meters (180 ft), whereas the geotechnical design indicated that piles cou be stopped at about 30 meters (100 ft) Pile Driving Analyzer (PDA) testing was done durin the installation of production piles to ensure that geotechnically derived design capacity valu were achieved and thus optimize the depth of pile driving This paper addresses details of th site characteristics, the details of the foundation investigation and pile driving analyzer te results The results of the PDA testing confirmed the geotechnical recommendations.

Introduction

Battle River is a very long river flowing mostly in an East-West direction in the central pa of the Province of Alberta Many highways, both Primary and Secondary, cross this rive Secondary Highway (SH) 855 is one such highway crossing the Battle River, approximately at a distance of 350 km southeast of Edmonton, the Provincial Capital of Alberta Since the existing river channel was very close to the valley wall, the highway was constructed initially along a sidehill alignment with substandard horizontal and vertical curves. The north side of the Battle River Valley was also quite famous for coal mining activity for a long time

A single lane lattice girder bridge with wooden deck planking provided the bridge structure facilitating crossing this river The highway was highly accident prone for many years especially during winter months due to icing conditions developing in the ditches from the seepage coming out of the coal mine excavated areas.

A coal fired electricity generating station also exists at this location In order to meet the cooling water requirements of the generating station, a small reservoir exists in the main channel of the river on either side of this old bridge. Because of the circulation of hot and cold water within the reservoir, the Battle River never freezes at this location A vicinity map in Figure 1 shows the location of SH 855 near Forestburg

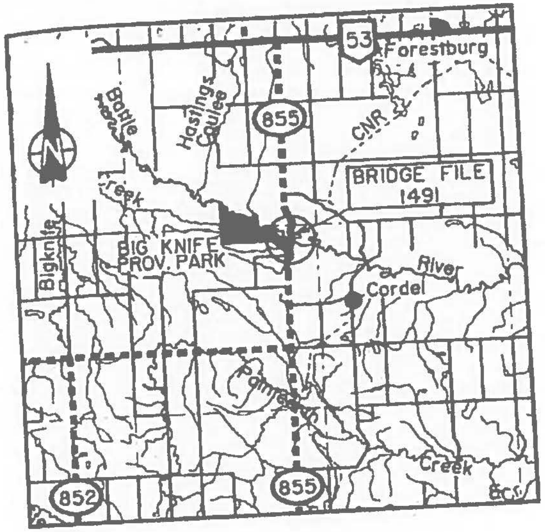

Figure 1. Vicinity Map

With increased traffic activity along this rural highway and occurrence of serious accidents especially during winter months, the Roadway Planning Branch of Alberta Transportation in 1982 initiated a functional planning study for realigning the highway within the Battle River Valley and the construction of a new two lane bridge

Different alignment options were considered through air photos, site inspections and input from different stakeholders within the Department It was also decided to provide a new channel by partially excavating the south flood plain area of the river and partially backfilling of the existing channel The new bridge would be located over the new channel Figure 2 shows an air photo of the site with relevant details of the location of old bridge, the proposed new highway alignment and location of the new channel

Foundation Conditions - Old Bridge Site

Historical records of the old bridge site indicated existence of loose silty sand (with moisture contents in the range of 35%) to a depth of approximately 60 meters at which depth shale/sandstone formations were identified However, existence of thin clay layers and gravel was also indicated within the broad spectrum of silty sand, reflecting indirectly the sedimentary nature of the subsurface material over a long period of time, geologically speaking. This historic information was also confirmed from the preliminary geotechnical investigation undertaken in 1983 at both ends of the old bridge as part of the realignment study.

Figure 2. Airphoto of Project Site

Geotechnical Investigation - Proposed Bridge Site

Field investigation was undertaken by the Geotechnical Section of Alberta Transportation in March-April 1990 in the flood plain area where the proposed new channel and the new bridge would be located. However, because of water in the existing channel, drilling could not be carried out at the proposed location of the north abutment of the bridge. Instead, drilling was done at the base of the north hill where the north side approach fill would be located.

The general stratigraphy in the Battle River flood plain consists of loose to compact silty sand deposit to a depth of about 16 meters beyond which the soil layers varied alternately between clay and sand formations. The depth of shale bed rock was nearly 63 meters. Water bearing layers were also noticed within the clay and sand formations. Standard Penetration Test blow counts were low (N<20) within the top 20 meters and gradually increased with depth below 20 meters.

An interesting feature of the ground characteristics was the observation of soil setup during the geotechnical investigation. On one or two occasions the drill rig broke down during the investigation and the drill rods and split spoon were left in the ground. The next day when the rig was operational and the investigation continued, it was very difficult to move the drill stem. In comparison with the blow counts during continuous operation, the blow counts the following day were found to be considerably higher indicating refusal conditions. In such instances, the drill stem had to be removed and the depth had to be advanced by auguring.

This observation demonstrated the known observation that during disturbance the sand would lose strength by increase in porewater pressure but that strength would be generally regained as the porewater pressure dissipated. A similar situation would be expected during pile driving, and as such it was important that the driving of the pile should be done in a continuous operation to the desired depth of embedment. A typical cross section of the proposed bridge configuration and the stratigraphy of the subsurface condition of the Battle River flood plain is presented in Figure 3.

Test Piles Driving

As part of conventional practice within the Department at important bridge sites, driving of test piles was undertaken in the flood plain area near the south abutment almost at the same time as the geotechnical investigation was done. The test piles included two 310 mm by 94 kg H-piles and two 410 mm diameter open end pipe piles. The objectives of the test pile driving are primarily (a) to determine the depth to which piles could be driven to reach refusal, (b) to determine the load capacity of the piles at various depths of embedment using the Hiley Pile Driving formula (Peterson, 1977) and (c) to assess, in a general way, what, if any, problems would likely be encountered during production piling.

Out of the four test piles, two were driven to a depth of about 34 meters and the other two, Test Piles 1 and 4 were driven to refusal. The hammer used for driving the piles was a Berta 1500 single acting diesel hammer with a rated maximum energy of 40.6 kJ. The number of blows required to drive each pile a depth of 250 mm was recorded. Interestingly, low blow counts (N < 20) were recorded up to a depth of 25 meters and from thereon higher values were recorded. It was also noted that although the depth of shale was in the order of 60-70 meters at the location of the test piles, the test piles met high resistance well above that depth (34 meters for H-piles and 45 meters for pipe piles) in the silty sand matrix.

Figure 4 shows a plan view of the approximate locations of the test piles in relation to the location of the new channel and the proposed new bridge and subsurface stratigraphy information close to the test piles location. Figure 5 shows typical plots of depth versus blows for test piles 1 and 4.

Except for the H-piles, the sizes of pipe piles used for test pile driving are not necessarily the same that would be used for production piling. In most cases, the test pile driving operations are routinely done before the geotechnical investigation and evaluation of the site is undertaken. The use of the Pile Driving Analyzer (PDA) for load capacity determination was not part of the routine test piling program undertaken by the Department.

Important Design Details

For supporting the bridge structure, steel H-piles were selected for the abutments and pipe piles for the piers as is customary for many structures. The piles proposed for the piers were 914 mm diameter open-ended pipe piles in-filled with concrete. Each of the six piles comprising the pier were designed to accommodate an extreme design load (Combination 4) of 2030 kN (203 tonnes) (CAN/CSA- S6-88, 1988).

During periodical in-house discussions between the Bridge Branch and the Geotechnical Section, two important geotechnical recommendations were made. The first

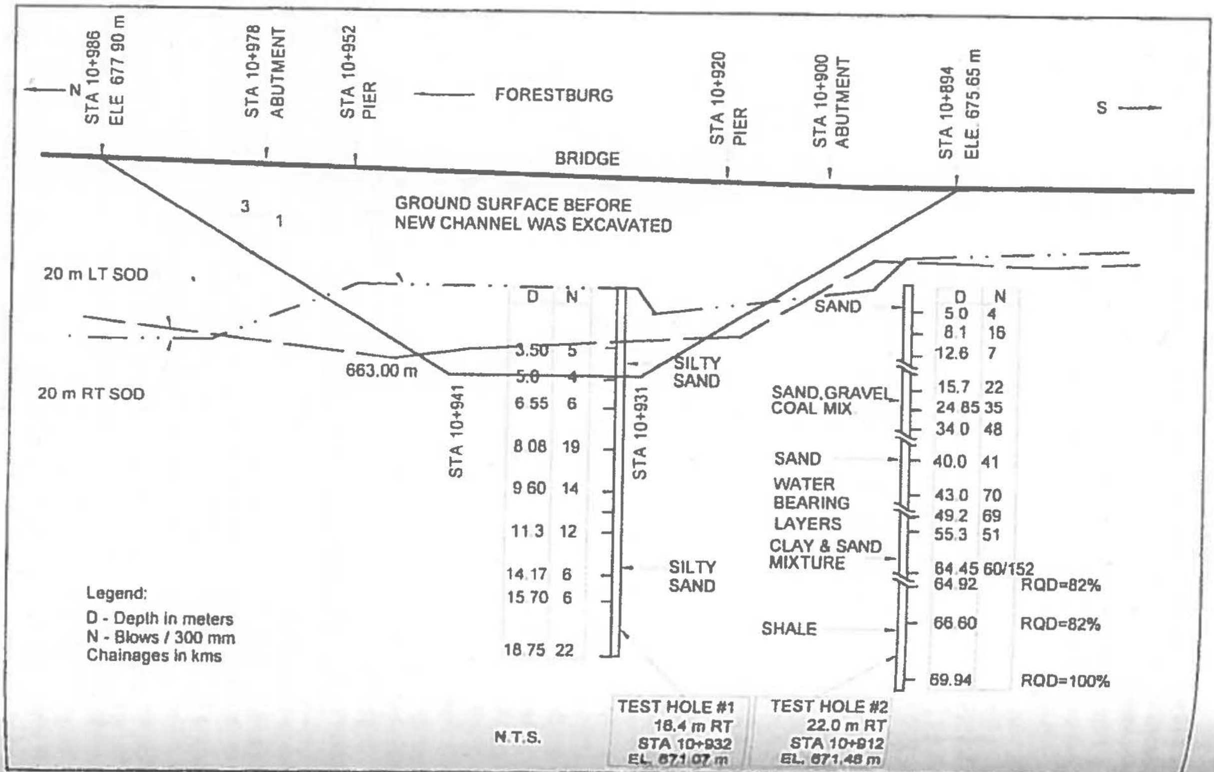

Figure 3 Typical Soil Stratigraphy at Bridge Site

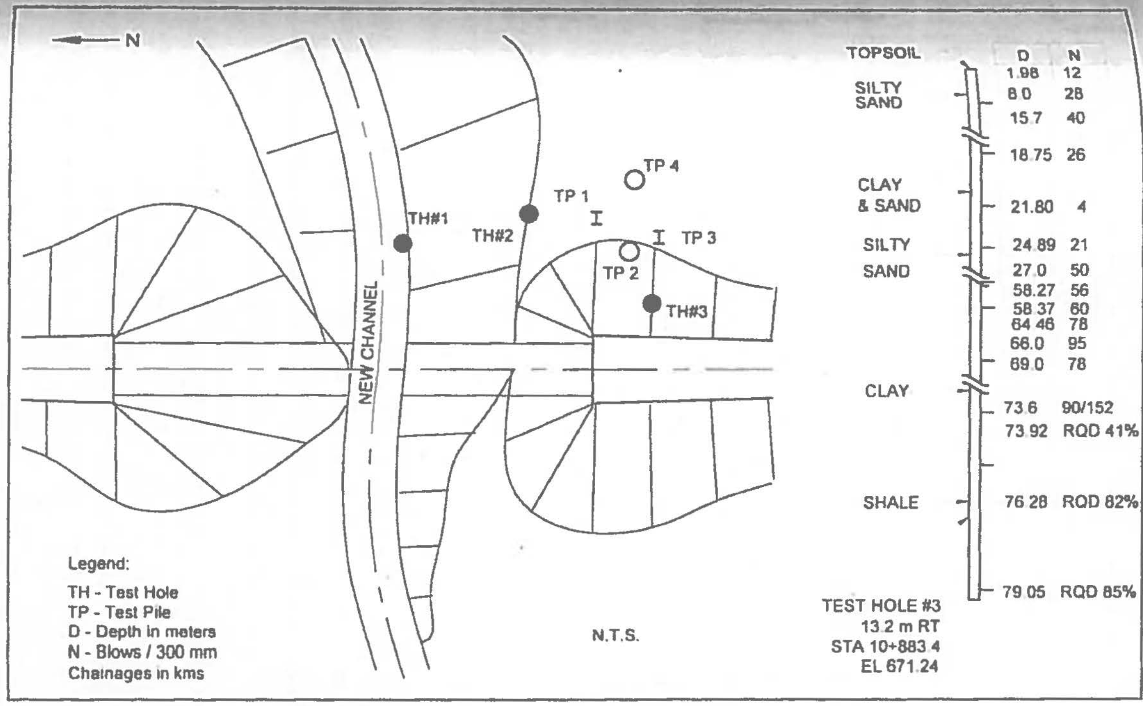

Figure 4. Location of Test Piles

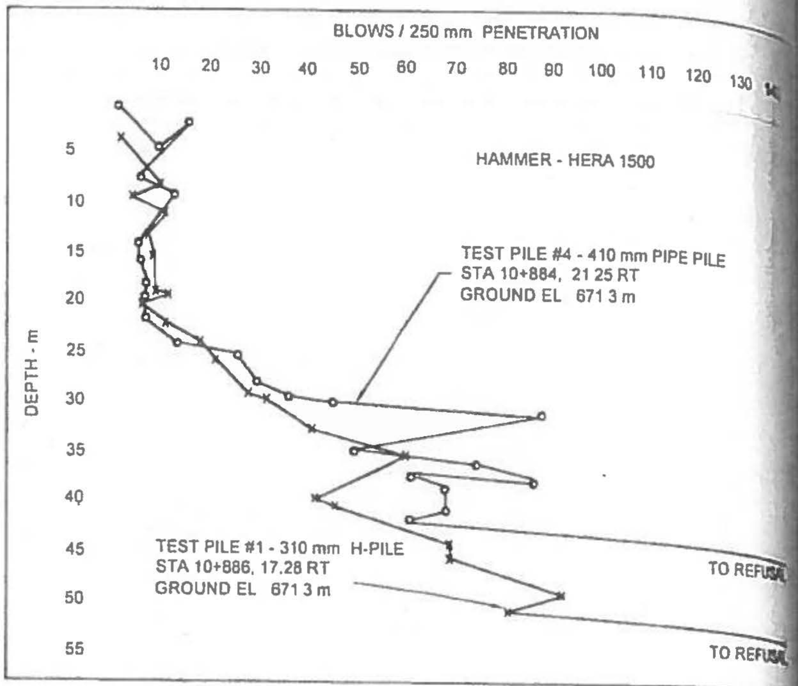

Figure 5. Test Piles Driving Record

one was to use granular material to backfill the old channel to minimize build up of exc pore pressures in the old channel location when the north side approach fill was construct over that area after the backfilling The second one was with regards to the depth of p foundations Instead of driving the piles to refusal, it was recommended based on sta analysis that the pile tips can be stopped at a higher elevation where the blow counts show an increasing trend Accordingly, a tentative elevation of 635.0 m was suggested for the pi tips for preliminary planning and design considerations. This recommendation was ma based on the field observations during the geotechnical investigation which indicated that e excess pore pressures generated due to pile driving would disperse quickly due to th predominant sandy nature of the subsurface material and hence frictional resistance wou also be rapidly regained along the shaft circumference of the piles

Since it was customary to drive piles to refusal, the recommendation of stopping th piles at a shallower depth was questioned by the structural engineers in relation to th adequacy of the capacities at the recommended depth. To verify that the design pile capaci would be achieved at the proposed depth of embedment, it was also recommended that PDA testing should be undertaken as this approach was known to provide fairly reliable resul (Diyaljee and Pariti, 2002) This testing was approved and was scheduled during th

installation of the production piles which were to be driven to the depth recommended in the geotechnical report

Construction of the New Channel

Excavation for the diversion channel was done with a dragline during the winter months (February/March) of 1990 Backfilling of the old channel was undertaken later during the summer of 1991 This backfilling was done with 75 mm maximum size pitrun gravel, and sand from nearby gravel pit sources

Installation of Pile Foundations

The bridge piling contractor undertook the pile driving operations between December 1991 and January 1992 For the installation of the pipe piles at the pier locations, a berm was pushed into the channel for access to the pier locations

Installation of the pipe piles for the North Pier (Pier #2) was the first to be done in December 1991 Interestingly, these piles reached refusal at around elevation 640 m (5 meters above the tentative elevation recommended for the preliminary design considerations) A quick evaluation of the geotechnical capacities for the piles was done in-house using conventional static analysis as well as by Fellenius's Method taking into the effect of downdrag and development of the neutral plane concept (Canadian Foundation Engineering Manual, 1985) These analyses gave ultimate pile capacities of 9090 and 6910 kN, respectively and factored pile capacities of 3636 and 2764 kN, corresponding to a capacity modification factor of 0 4 (CAN/CSA-S6-88) Since these values were in excess of the extreme loading condition, it was decided that this higher elevation was satisfactory for terminating the piles

Pile Driving of South Pier and PDA Testing

The PDA testing was done on two production piles of the south pier pile group (Pier #1) The testing was coordinated to be within the work schedule of the Contractor's operations so that no delay would be caused to his work operations

Anna Geodynamics of the City of Ottawa, Ontario, Canada, carried out the testing on January 21, 1992 with a re-strike the following day The testing was performed to assess the static bearing capacity at End-of-Initial Driving (EOID) and at Beginning-of-Restrike (BOR)

The two production piles tested were 1H 25V inclined 914 x 9 5 mm open-toe steel pipe piles marked as #3 and #4 of the south pier pile group Prior to testing, strain gauges and accelerometers were attached to the two test piles in accordance with the standard procedure outlined by the Consultant (Cheng and Weins, 1992) The two piles were installed to an average embedment depth of 30 m prior to testing corresponding to a tip elevation of 638 m The pile driving hammer used was a Berminghammer B-400 single-acting diesel hammer Typical photographs taken at the time of the PDA test are included in Figures 6 through 8

Piles 3 and 4 were monitored with dynamic measurements on January 21, 1992 for the last 1.2 m (48") of penetration and were subsequently restruck on January 22 after a waiting period of 20 hours A total of 12 blow records of the two piles were processed for the purpose of analyzing the PDA testing results (Fellenius, 1992).

Although rated with a nominal energy of 61 kJ, the energy transferred by the B-[illegible] hammer ranged from about 17 kJ through 22 kJ The ratio of transferred energy to no[illegible] energy ranged from about 25% through 35 % A summary of dynamic measureme[illegible] shown in Table 1.

Activated Pile Capacity. The activated static resistances for both piles were evaluated [illegible] the CMES-RMX approach and a J-factor of 0 6 (Table 2) For Pile 3, the maximum acti[illegible] static resistance at End-Of-Initial-Driving (EOID) was about 2725 kN for a penetr[illegible] resistance (PRES) of 40 blows/25 mm At Beginning-Of-Restrike (BOR) the max[illegible] activated resistance was about 2795 kN for a PRES of 45 blows/25 mm On the other [illegible] for Pile 4, the maximum activated static resistance was about 2790 kN for a PRES [illegible] blows/25 mm at EOID and 3170 kN at BOR for a PRES of 54 blows/25 mm

The activated static resistance values of Pile 4 were also calculated by using CAP[illegible] analysis and were found to be 2920 kN at EOID and 3220 kN at BOR, slightly higher [illegible] those predicted by CMES - RMX approach

Figure 6. Attaching Strain Gauges and Accelerometers to Pile # 3

TABLE 1. SUMMARY OF DYNAMIC MEASUREMENTS

Pile and Blow#	EMBD (m)	EMAX (kJ)	ENERGY RATIO (%)	FIMP (kN)	SIMP (MPa)	FMAX (kN)	SMAX (MPa)	CMES (kN)	PRES (bl/25 mm)
Pile 3 (Length = 34.3 m)									
Initial Driving									
[illegible]-ID	30 3	20.6	35	4000	150	4000	150	2600	20
[illegible]-ID	31 3	19.4	30	3660	135	3670	135	2650	34
[illegible]-EOID	31 5	20.0	35	3595	135	3595	135	2725	40
Restrike									
3-BOR	31 5	16.5	25	3565	130	3565	130	2795	45
5-RSTR	31 5	20.1	35	3820	140	3830	140	2835	45
10-EOR	31 5	18.8	30	3670	135	3670	135	2820	45
Pile 4 (Length = 33.7 m)									
Initial Driving									
[illegible]-ID	30.0	20.0	35	3535	130	3590	135	2790	25
[illegible]-ID	30.3	20.8	35	3525	130	3555	130	2725	25
[illegible]-EOID	30.9	19.5	30	3365	125	3365	125	2715	45**
Restrike									
3-BOR	30.9	22.0	35	4015	150	4015	150	3170	54**
4-RSTR	30.9	20.2	35	3945	145	3945	145	3070	54
10-EOR	30.9	19.3	30	3595	135	3595	135	2940	54

Length: Length of pile below dynamic monitoring gages
EMBD: Embedment depth is considered to be at surface water level
EMAX: Maximum transferred energy
FIMP: Impact force
SIMP: Impact stress
FMAX: Maximum compressive force
SMAX: Maximum compressive stress
CMES: Case Method Estimate of bearing capacity using RMX method with J-factor 0 6
PRES: Equivalent penetration resistance from penetration measured for a series of blows
ID: Initial Driving
EOID: End-of-Initial-Driving
BOR: Beginning-of-Restrike
RSTR: Restrike
EOR: End-of-Restrike
**: Blow selected for CAPWAP analysis
Note: The energy ratio is the ratio of transferred energy to nominal energy of the hammer (Nominal energy is 61 kJ as per GRLWEAP hammer files)

TABLE 2. SUMMARY OF ACTIVATED STATIC RESISTANCE

PILE AND BLOW #	DESIGN LOAD		ACTIVATED STATIC RESISTANCE				RESISTANCE RATIO	PRES (Blows/ 25 mm)
			CMES		CAPWAP			
	kN	Tons	kN	Tons	kN	Tons		
PILE 3								
1529 - EOID	2000	225	2725	307	--	--	1.4	40
3 -BOR	2000	225	2795	314	--	--	1.4	45
PILE 4								
1271 - EOID	2000	225	2790	314	2920	328	1.4	45
2 - BOR	2000	225	3170	357	3220	362	1.6	54

CMES	Case Method Estimate of bearing capacity using RMX method with J- factor of 0 6
PRES	Penetration resistance calculated from the penetration measured for a series of blows
EOID	End of Initial Driving
BOR	Beginning of Restrike

Resistance Ratio: Ratio of the activated resistance (CMES) to the design load

The values of shaft resistance at EOID and BOR were also evaluated for Pile 4 using CAPWAP analysis (Tables 3 and 4) and were found to be 2240 kN and 3080 kN respectively showing an increase in shaft resistance of about 40% over a 20 hour period between the two stages The corresponding toe resistance computed at EOID and BOR were 680 kN and 1[illegible] kN, respectively, which indirectly indicated that the toe resistance was not fully engaged. This was also similar for Pile 3 based on a qualitative evaluation of the measured force and velocity graph.

The increase in penetration resistance from 45 blows/25 mm at EOID to 54 blows/25 mm at BOR was accounted for by the increase in skin friction This being the case, the increased energy was expended in overcoming the frictional resistance and would not have been sufficient to allow movement of the pile toe as would have occurred at the EOID when the pore water pressures were higher The lower toe capacity reported at BOR would therefore be plausible. This finding also confirms the observation of soil set up made during the drilling operation that if the operations were stopped it was difficult to advance the drilling tools the next day by driving

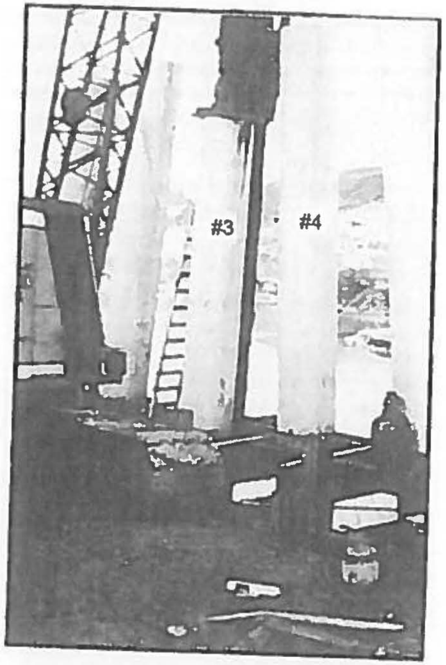

Figure 7. Pile Driving in Progress

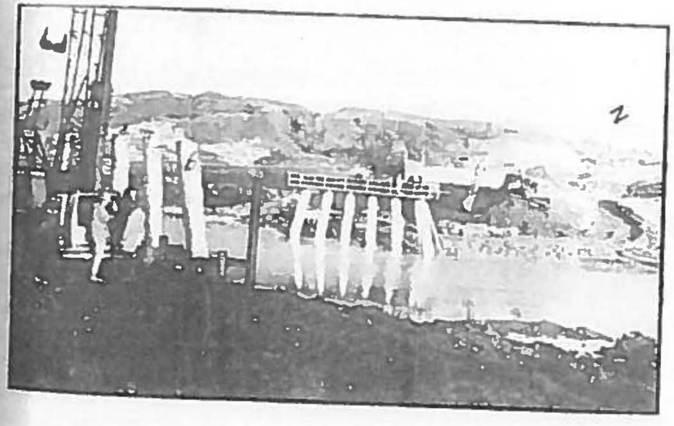
Figure 8. General View of Pipe Piles of South and North Piers

The PDA testing results were also analyzed to determine a more representative capacity at BOR assuming that the resistance at the toe of the pile remains unch between EOID and BOR This assumption yielded a value of 3760 kN for the activ static bearing capacity at BOR (shaft resistance at BOR + toe resistance at EOID) Since penetration resistance results from the driving records during the actual driving of production piles were exceedingly high, it was felt that the computed capacities represe lower bound estimates of the actual pile capacity

In order to determine the likely capacity of the piles beyond the CAPWAP activ values, static analyzes were also undertaken using the UNIPILE program (Godreault Fellenius, 1990). This program calculates pile capacities based on the effective stress friction ratio (Beta-method) From this analysis it was determined that the capacity of the p was 4100 kN utilizing Beta coefficients ranging from 0.25 to 0.28 and hydrostatic pore distribution

The friction ratio coefficients and the toe bearing coefficient (N_t) were chose match the capacity results from dynamic measurements and CAPWAP analyses. Th resistance of 4100 kN was made up of 3100 kN shaft resistance and 1000 kN toe resista By further applying soil parameters that are usually expected for the soil conditions at the - Beta values of 0 25 to 0 4 and toe resistance coefficient of 140, a capacity of 6000 kN very likely for this site This capacity was within the range of values obtained from the analyses done at the design stage and reported previously, and hence did not seem unrea

In summary, the depth of pile embedment recommended in the geotechnical rep was maintained, since the pile capacities obtained by various methods from the analysis of PDA testing results exceeded the design load requirement The PDA testing demonstrated that there is a need to select the pile driving hammer with reference to w equation analysis so that the pile can be moved to engage both friction and toe resista The use of pile hammers delivering inadequate energies seems to be an often encounte problem in PDA testing

Discussion

Very often geotechnical engineers are faced with determining depths of pile embedment f which pile capacities are determined In order to determine where piles would termin experience, understanding of soil stratigraphic characteristics, soil behavior during drivi and hammer characteristics are required to estimate the depth at which driven piles wo terminate. This depth can be reasonably estimated by experienced geotechnical engine based on a careful study of the ground conditions and utilizing past experiences from proje which they have designed and monitored the piling operations

In some cases test piles are driven and the depth of refusal of such piles is often u as the depth of termination of the production piles However, it has been noted that so soil deposits with high ground water levels and saturated conditions, that would norm show dense characteristics would show very little resistance during the driving of test pi or production piles and during the driving of the split spoon during standard penetra resistance (SPT) testing Considerable savings can be realized if piles can be terminated shallower depth This determination is one of the challenges facing the geotechnical des engineer in undertaking pile design as the values for conventional static analysis is d

pendent on the measurement of the in-situ properties which can be influenced by the test thod used As noted the SPT test can provide values in sands that reflect subsoil conditions a disturbed state.

The Battle River site was one which proved that careful understanding of soil acteristics and their behavior can be utilized with confidence to support the geotechnical signs made In this case, the recommendations made from the static analysis were firmed by the PDA testing which is generally widely utilized in present day practice to ermine capacity as piles are advanced This information allows decisions to be made on bedment depths on site

TABLE 3: CAPWAP Analysis Results for Pile 4 at EOID. (Blow No. 1271)

Soil Segmt No.	Depth below gages (m)	Depth below grade (m)	Ru kN	Sum of Ru UP kN	Sum of Ru DOWN kN	Unit Resistance with respect to Depth kN /m	Unit Resistance with respect to Area kN/m2	Smith Damping (s/m)	Quake (mm)
				2918.7					
1	5.1	2 3	66.2	2852.5	66.2	32.43	11.30	0 321	1.0
2	7 1	4 3	113.5	2739.0	179.7	55.56	19.36	0 321	1.0
3	9 2	6 3	168.3	2570.8	348.0	82 39	28.71	0 321	1.0
4	11 2	8 4	183.9	2386.8	531.9	90.05	31.38	0 321	1.0
5	13.3	10 4	179.0	2207 8	710.9	87.65	30.54	0 321	1.0
6	15.3	12 5	179.0	2028 8	889.9	87.63	30.53	0 321	1.0
7	17.4	14 5	139.7	1889 1	1029.6	68.40	23.83	0 321	1.0
8	19.4	16 6	67.7	1821.4	1097.3	33.16	11.55	0 321	1.0
9	21,4	18 6	27.0	1794.4	1124.3	13.22	4.61	0 321	1 0
10	23.5	20 6	36.7	1757.7	1161 1	17.99	6.27	0 321	1.0
11	25.5	22 7	92.5	1665.1	1253.6	45.31	15.79	0 321	1.0
12	27.6	24 7	184.2	1481.0	1437.8	90.17	31.42	0 321	1.0
13	29.6	26 8	301.7	1179.3	1739.4	147.70	51.46	0 321	1.0
14	31.7	28 8	277.3	902.0	2016.7	135.75	47.30	0 321	1.0
15	33.7	30 8	223.9	678.1	2240.6	109.63	38.20	0 321	1.0
Average Skin Values			149 4			72 63	25.48	0 321	1 0
Toe			678 1				1033.75	1 142	0 0

Final CAPWAP CAPACITY: Ru = 2918.7 kN. Skin = 2240.6 kN, Toe = 678.1 kN

Soil Model Parameters / Extensions	Skin	Toe
Case Damping	0.650	0.700
Unloading Level (% of Ru)	0.0	0.0
Unloading Level (% of Ru)	1	
Soil Plug Weight (kN)		1.50

TABLE 4: CAPWAP Analysis Results for Pile 4 at BOR. (Blow No. 2)

Soil Segmt No.	Depth below gages (m)	Depth below grade (m)	Ru kN	Sum of Ru UP kN	Sum of Ru DOWN kN	Unit Resistance with respect to Depth kN /m	Unit Resistance with respect to Area kN/m2	Smith Damping (s/m)	Qu[illegible]
				3223.3					
1	5.1	2.3	142.2	3080.1	142.2	69.60	24.24	0.324	[illegible]
2	7.1	4.3	236.4	2843.7	378.6	115.75	40.32	0.324	[illegible]
3	9.2	6.3	215.6	2628.1	594.2	105.57	36.77	0.324	[illegible]
4	11 2	8.4	81.7	2546.4	675.9	40.01	13.94	0.324	[illegible]
5	13.3	10.5	64.3	2482.1	740.2	31.47	10.96	0.324	[illegible]
6	15.3	12.5	199.0	2283 1	939.2	97.44	33.94	0.324	[illegible]
7	17.4	14.6	272.9	2010.2	1212.1	133.63	46.54	0.324	[illegible]
8	19.4	16.6	177.6	1832.5	1389.7	86.96	30.29	0.324	[illegible]
9	21,4	18.6	31.9	1800.7	1421.6	15.59	5.43	0.324	[illegible]
10	23.5	20.7	0.0	1800.7	1421.6	0.0	0.00	0.324	[illegible]
11	25.5	22.7	54.9	1745.8	1476.5	26.88	9.36	0.324	[illegible]
12	27.6	24.8	207.7	1538.1	1684.2	101.69	35.42	0.324	[illegible]
13	29.6	26.8	541.0	997.1	2225.1	264.87	92.26	0.324	[illegible]
14	31.7	28.9	626.9	370.2	2852.1	306.96	106.92	0.324	[illegible]
15	33.7	30.9	227.7	142.5	3079.8	111.50	38.83	0.324	[illegible]
Average Skin Values			205 3			99 67	35 01	0 324	1.30
Toe			142 5				217 17	6 212	0.5

Final CAPWAP CAPACITY: Ru = 3222.3 kN. Skin = 3079.8 kN, Toe = 142.5 kN

Soil Model Parameters / Extensions	Skin	Toe
Case Damping	0 903	0 800
Reloading Level (% of Ru)	0 0	0 0
Unloading Level (% of Ru)	0 0	

This situation is generally of valued interest to industrial sites where a large number of piles may be utilized. For bridge structures while this is applicable, it is often argued that for the cost of testing, an additional number of piles can be driven and additional capacity obtained. Such an argument is applicable for situations where, for example, a Highway Department purchases piles in bulk from a manufacturer or supplier and provides these to the Contractor. In this situation, all that the Contractor is paid for is the cost of driving the piles to a certain depth.

On the other hand, if the Contractor has to supply and place the piles, then the need to know where piles should terminate at depth becomes of increasing importance to the Highway Department, as driving long piles would be costlier since the Contractor has to purchase the piles. The PDA testing would no doubt be beneficial to contracts written in this context.

Apart from the above, the necessity for the geotechnical and structural engineers to be confident ahead of time on the foundation design will undoubtedly lead to better and cost effective pile designs.

A word of caution is that in an active seismic zone, the approach used for this site may not necessarily be the best since one has to consider the behavior of the foundation soils in relation to earthquake loading which can lead to liquefaction of saturated silty sand deposits. For this particular case founding piles in the shale foundation stratum may be more desirable.

Conclusion

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

- Driving foundation piles to refusal to attain desired pile capacities is not always necessary provided a detailed evaluation of the results of the field investigation is undertaken.
- In-situ soil characteristics are likely to be influenced by the nature of testing. Where loosening of the soil matrix of predominantly sandy soils in river environments is likely, the blow counts of the standard penetration test may not be accurate.
- PDA testing allows confirmation of the evaluation of the load capacity and depth of embedment of the piles determined from a combination of static analysis, experience and engineering judgement.
- The results of PDA testing are influenced by the energy output by the pile hammer as the true capacity of the pile is a function of skin frictional and end bearing resistances. Improper hammer energy may not allow the pile toe to be moved which would lead to lower pile capacities.
- It would be desirable to utilize test piles that are sized for production piles to undertake test pile evaluation using the PDA since this information can be used to assist in finalizing pile design before the start of production piling.
- PDA testing on small size piles can be helpful in arriving at parameters which can be extrapolated to estimate the capacity for the production piles.

Acknowledgments

The authors would like to thank the authorities of Alberta Transportation for allowing them to make use of the data in the Department files for preparing this paper. The opinions expressed in the paper are entirely those of the authors and may not necessarily constitute a position of the Department. Thanks are also due to the Airphotos Section, Alberta Environmental Protection, for allowing the reproduction of a copy of the airphoto of the project site.

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