High strain dynamic pile testing in a challenging soil condition – A case study in Southern Alberta, Canada

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ABSTRACT
Driven piles are increasingly utilized as foundation elements in challenging geotechnical conditions that have where drilled shafts would have been deemed appropriate. Predicting driven pile performance and capacity can be a daunting task. Unique soil conditions or variation across the subsurface of a project site can add further challenges to predicting performance of driven piles. High strain dynamic pile testing provides a fast, effective, and economical quality control tool for the installation of driven piles. This paper describes the application of High strain dynamic testing in performance evaluation of driven piles installed in a very dense sand ground condition in southern Alberta, Canada. The project site is located in the vicinity of Fort MacLeod, Alberta. Site conditions and pile installation process are described in detail. Pile instrumentation, test procedures, and PDA test results and their relevancy are discussed in detail. The results of CAse Pile Wave Analysis of Program (CAPWAP®) are discussed comprehensively in order to provide baseline conclusion that can be used in future for analysis of driven piles with similar soil conditions. A characteristic load settlement curve was developed for driven piles installed in very dense sand using the results of CAPWAP analysis performed on eighty two piles.

RÉSUMÉ
Les pieux battus sont de plus en plus utilisés comme éléments de fondation dans des conditions géotechniques difficiles où leurs pendant vissés auraient historiquement été considérés appropriés. Prédire les performances du pieu et sa capacité peut s'avérer une tâche ardue. Des conditions uniques ou une grande variabilité des propriétés du sol d'un site peuvent significativement complexifier la prédiction des performances. Les essais de chargement dynamiques de pieux à grande déformation fournissent une méthode de contrôle rapide, efficace et économique lors de l'installation de pieux battus. Ce document décrit l'utilisation de l'analyseur de battage de pieux (PDA) dans l'évaluation de la performance des pieux battus installés dans un sol en argile raide dans le sud de l'Alberta, Canada. Le site du projet est situé dans les environs de Fort MacLeod, en Alberta. Les conditions du site et le processus d'installation des pieux sont décrits en détail. L'instrumentation des pieux, les procédures d'essai et les résultats des tests PDA et de leur pertinence sont discutés en détail. Enfin, une analyse CAse Pile Wave, réalisée en utilisant le logiciel CAPWAP®, est présentée en détail afin de fournir une conclusion de référence qui pourra être utilisée pour l'analyse des pieux battus dans des conditions de sol similaires.

1 INTRODUCTION

High-strain dynamic testing of piles (ASTM D4945) has been conducted for more than 40 years. The first pile driving analyzers (PDA) were simple analog computers without storage capability. Over the past few decades significant advancement has been made on the understanding of and testing of deep foundations. PDAs have also evolved considerably and are now made with flexible digital microcomputers with significant internal storage capacity, wireless connectivity, and graphic display. However, even though some new parameters have been added, the basic equations used to calculate the bearing capacity of the test piles are still the same.

The concept of this testing procedure is to apply massive dynamic impact on the element and convert the resultant into static capacity. The basic purpose of high strain dynamic pile testing is to evaluate pile static capacity and its structural integrity using measurement of both force and velocity. CAse Pile Wave Analysis Software (CAPWAP) is utilized to analyze the field data (interpret the dynamic wave) and conduct the conversion into the required pile capacity.

The method involves attaching a minimum of two pairs of strain transducers and accelerometers on diagonally opposite sides of the pile. These are fixed minimum two (2) times the pile diameter below pile top. Impact is generated using a hammer capable of delivering the required impact to achieve the target load. The load and height of drop is pre-calculated using the soil parameters using GRL Wave Equation Analyses and Drivability Studies (WEAP) analysis.

The strain transducers attached to the pile measure the strain on the piles during impact, whereas accelerometers record the accelerations generated in the pile.

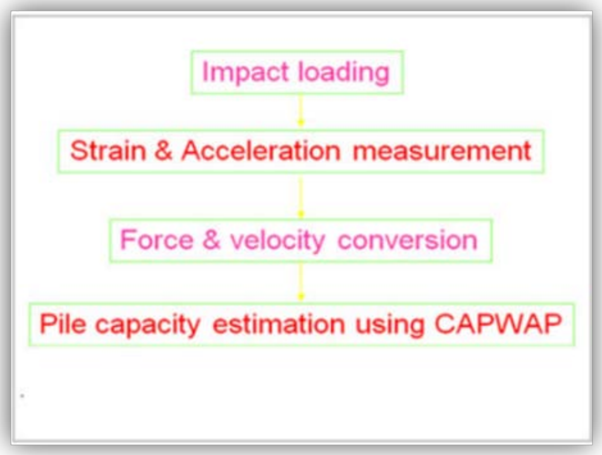

Figure 1. Typical procedure in estimating static pile capacity using high strain dynamic testing.

The benefits of high strain dynamic testing include:

- Significantly faster and more economical than static testing.
- Very effective for large diameter bored piles with huge capacities.
- Applicable for both offshore and onshore piling.
- Very effective for offshore piles, where conducting static load testing is very difficult.
- Applicable to most pile types.
- Is not restricted by space constraints.
- Provides reliable information on friction, end-bearing, pile integrity, hammer performance and plot simulated load test curve for comparison with static testing.
- Reduces controversies and eliminates risk enabling a safe foundation.
- Measures driving stresses for effective driving control.

The limitations of high strain dynamic testing include:

- Requires engineering and experienced judgment.
- The resistance to driving generated should be strong enough to mobilize pile ultimate resistance.
- Reliability and accuracy of the results are related to the proficiency of the setup as well as the hammer drop.

The soil condition encountered at the site consisted of dense to very dense sand and gravel. As a result, direct pile driving at the site was determined to be challenging with the potential for pile damage and bending. As such, it was decided during design phase to use predrilling techniques to install the piles at the site. Predrill holes were advanced at 90 percent of shaft diameter to the depth required depth. The predrill holes were terminated a minimum distance of 1.5 m or 3 times the pile shaft diameter, whichever is greater, above the pile tip elevation.

2 SITE DESCRITPTION

The geotechnical site investigation was performed and recommendation was provided by SNC Lavalin (SNC Lavalin, 2014). The project site is a new electrical substation intended to support the distribution of power generated from a nearby wind farm. Approximately 800 driven piles were to be installed to support the proposed development. The legal description of the substation is SW-17-08-26-W4M. This location is north of Township Road 82 and east of Range Road 265 which is southwest of Fort MacLeod, Alberta, as shown in Figures 2 below. Prior to construction, the site was undeveloped farm land and relatively flat.

Figure 2. Project location of windy flats substation near Fort MacLeod, AB

Geological evidence and published geology maps indicate that the surficial strata is related to Pleistocene Moraine till, consisting of an unsorted mixture of clay, silt and sand as well as minor pebbles, cobbles, and boulders. Locally, the Moraine till may contain blocks of bedrock, stratified sediment, or lenses of glaciolacustrine and/or glaciofluvial sediment. Bedrock (Willow Creek Formation) in the area is relatively shallow. The Paleocene and upper cretaceous Willow Creek Formation is pale grey, fine-grained, calcareous sandstone, thick bedded and coarse grained in upper part; grey, green and pink bentonitic mudstone with abundant whiteweathering calcareous concretions; scattered thin limestone beds; non-marine.

Based on the material encountered in the boreholes, the general soil profile consisted of topsoil over variable thickness of glacial till (to depths of approximately 1.68 to 3.05m below ground surface followed by varying sand/gravel deposits to the final depth of most boreholes. Inter/intra glacial till (sand/silt) deposits were encountered within the sand/gravel layers in most of the deep boreholes. The thickness of topsoil varied from 80 mm to 180 mm and 20 mm to 130 mm at the substation and access road areas, respectively.

The till deposits were typically stiff to hard, low to medium plasticity and dry to moist. The sand/gravel deposits were generally dense to very dense, fine to coarse and dry, except few layers of compact sand/gravel at various depths in boreholes. The inter sand and silt till deposits varied in thickness between thin pockets to lenses up to 0.8 m in thickness. Typically these layers were very stiff (based on limited SPTs and drilling difficulty). Figure 3 shows the profile of SPT N values with depth for all substation boreholes.

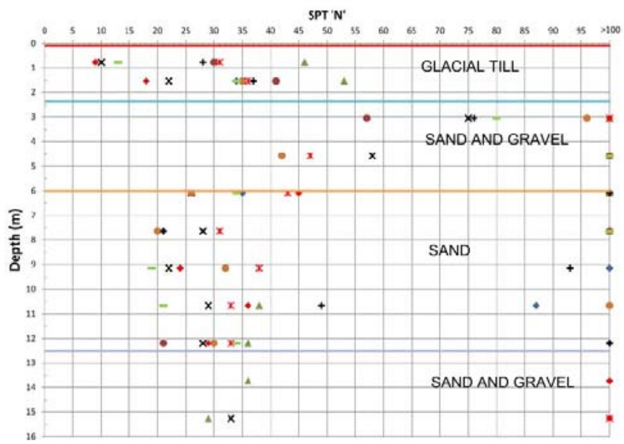

Figure 3. Variation of SPT blow count with depth at borehole drilled at the site

No seepage was observed in any of the boreholes. There was no groundwater encountered immediately after installation, all standpipes were dry.

Cobbles and coarse gravel were not directly noted during the drilling of the boreholes in the substation. Since ODEX drilling was used in the investigation, it may have reduced the size of gravel and cobbles before they are returned to the surface, therefore the true size of gravel/cobble pieces is only an estimate. The investigation report also indicated evidence of fresh fracturing on ODEX cuttings and from drilling difficulty in certain soil layers, coarse gravel and cobbles were expected at most locations. Some cobbles were noted at the ground surface during drilling and in select auger grab samples. Possible boulders were encountered within sand/gravel deposits as evidenced during the SPT rod bouncing during test execution. Cobbles and boulders are often located randomly within glacial till deposits but can also form sorted layers, such as boulder pavements. The actual location and frequency of cobbles and boulders varies and the probability of encountering such deposits increases with the number of holes drilled. Cobbles/boulders should be anticipated during the installation of foundations at this site. No groundwater was encountered during drilling.

3 DRIVEN PILE DESIGN

Driven, steel pipe piles have been designed on the basis of Limit state design. In accordance with the 2010 Edition of the National Building Code of Canada (NBCC 2010). Foundation designs using a limit state design approach should satisfy the following design equation, as taken from the 2006 Edition of the Canadian Foundation Engineering Manual (CFEM 2006).

$$\Phi R_n \geq \Sigma \alpha_i S_{ni} \qquad [1]$$

Where:

ΦR_n-Factored geotechnical resistance

Φ-Geotechnical resistance factor

R_n-Ultimate geotechnical resistance

$\Sigma \alpha_i S_{ni}$- Summation of the factored overall load effects for a given load combination condition

α_i- Load factor corresponding to a particular load

S_{ni}- Specified load component of the overall load effect, such as dead load due to weight of structure or live load due to wind

I-Various types of loads such as dead load, live load, wind load and so on.

The recommended geotechnical resistance factors for use in designing foundations in an ULS framework are given in NBCC 2010. According to NBCC, a higher resistance factor of 0.5 (for compressive loading) is prescribed for the design of deep foundation that appropriately utilizes high strain dynamic testing in design and construction of deep foundations.

The factored geotechnical resistance is determined by multiplying the applicable resistance factor and the unfactored ultimate geotechnical resistance. The foundation is considered acceptable relative to the ULS condition if the factored geotechnical resistance at ULS is greater than the factored structural load. In addition, a check must also was made to confirm that deformations or settlements of the foundations are acceptable under the serviceability limit state (SLS) loading conditions.

The ultimate shaft resistance values of the subgrade soils that are used in the design are presented in Table 1 below provides the geotechnical parameters utilized in the design.

Based on these parameters, the driven piles were designed to take the applied structural loading. Configurations of the designed piles and the corresponding loadings are provided in Table 2, Table 3, and Table 4 respectively. Serviceability Limit State Design Criterion used in the design of these piles were 6 mm, 25 mm for lateral vertical deflections respectively. It was also assumed during design that a hydraulic hammer with an efficiency of at least 70 percent would be used during construction to drive the piles.

Table 1. Geotechnical design parameter for pile design

Soil layer	Layer 1	Layer 2	Layer 3	Layer 4
Soil type	Glacial till	Sand and gravel	Sand	Sand and gravel
Depth below grade (m)	2.0	6.0	12.5	below 12.5
Effective weight (kN/m^3)	20	20	21	21
ULS shaft resistance (kPa)	-	70	100	115
ULS toe resistance (kPa)	-	-	3,000	10,000
Effective friction angle (deg)	30	34	32	34

Table 2. Driven pile design based on empirical geotechnical design parameters

Pile mark	P1a	P1b	P1c	P2a
Pile count	74	108	14	18
Specified loads	See Table 4			
Shaft diameter (mm)	324	406	508	406
Wall thickness (mm)	12.7	12.7	12.7	12.7
Projection (mm)	140 – 700 for all types			
Pile design length (m)	6.7	6.7	6.7	8.7
Min pile embedment	6.0	6.0	6.0	8.0
Termination criteria (Bl/250 mm)	4	4	4	7
Hammer energy (kJ)	50	50	50	50

Table 3: Driven Pile Design Based on Empirical Geotechnical Design Parameters

Pile mark	P2b	P3	P4	P5	P6
Pile count	128	12	24	108	36
Specified loads	See Table 4				
Batter angle (deg)	-	-	15	15	0
Shaft diameter (mm)	508	273	324	324	324
Wall thickness (mm)	12.7	12.7	12.7	12.7	12.7
Projection (mm)	140 – 700 for all types				
Pile design length (m)	8.7	10.7	12.1	15.2	15.2
Min. pile embedment (m)	8.0	10.0	11.4	14.5	14.5
Termination criteria (Bl/250 mm)	7	12	17	20	20
Hammer energy (kJ)	50	50	50	50	50

Table 4: Foundation Loads on Piles

Pile mark	Specified factored loads			
	C (kN)	T (kN)	$V_{x,y}$ (kN)	$M_{x,y}$ (kN-m)
P1a	74	-	14	52
P1b	38	-	28	48
P1c	40	-	29	98
P2a	36	-	28	48
P2b	87	21	35	115
P3	150	-	15	0
P4	521	438	104	191
P5	995	785	172	738
P6	995	785	172	738

4 HIGH STRAIN DYNAMIC TESTING

Traditional pile testing methods have significant disadvantages. Static load tests are expensive and time consuming and in many cases do not fit with the project schedules. Conversely, pile dynamic testing is a cost effective, fast, and highly reliable quality assurance method to indirectly estimate the pile capacities. In order to comply with the NBCC2010, the dynamic testing of piles must be performed according to the standards prescribed in ASTM D4945.

Research performed at CASE Western Reserve University (Rauche et al. 1985) formed the basis for the modern day high strain dynamic testing. There have been many studies done in the past to validate the applicability of high strain dynamic testing. These studies include correlation studies performed on the data derived from static and dynamic testing of deep foundations (Likins et al. 1996). High strain dynamic testing consists of estimating soil resistance and its distribution from force and velocity measurements obtained near the top of a foundation that is impacted by a hammer or drop weight. The impact produces a compressive wave that travels down the shaft of the foundation. Figure 4 shows the typical sensor setup for high strain dynamic testing that was used at site.

The PDA uses signals from strain and acceleration transducers which are bolted and anchored to the test pile to estimate the pile load carrying capacity. Using the strain and acceleration PDA utilizes closed form solutions for the hammer impact wave propagation in a pile to evaluate the following:

- Hammer performance to qualify pile driving equipment;
- Preliminary estimate of activated bearing capacity (CASE Method Estimate) during pile driving and /or during re-strike;
- Driving stresses to investigate potential damages in a pile;
- Assess effects of changes to the driving system; and

- Structural integrity of driven pile shaft.

Quasi instantaneous response from the PDA is displayed for each hammer impact offering the engineer multiple resources to monitor the test progress in real time. CAse Pile Wave Analysis Program (CAPWAP) combines measured force and velocity data with wave equation analysis to calculate the soil resistance forces acting on the pile.

Figure 4. Sensor setting for dynamic testing of helical piles at the site

Figure 5 shows the typical profile of a measured velocity and force in a pile.

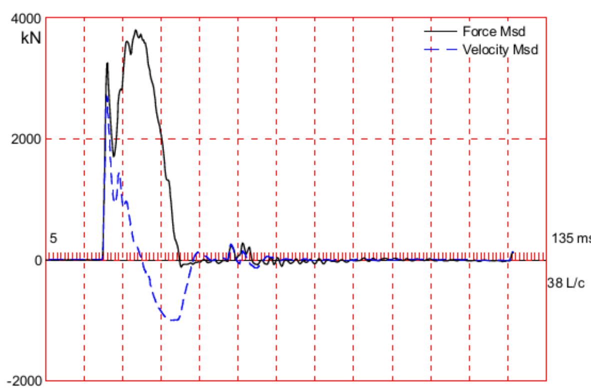

Figure 5. Measured force and velocity profile of a pile

A pair of strain transducers obtain the signal necessary to compute forces, while measurements from a pair of accelerometers are integrated to yield velocity. These sensors are connected to an instrument (such as a pile driving analyzer ®), that records, processes and displays data and results.

As long as the wave travels in one direction, force and velocity are proportional and related by the equation (2).

$$F = Zv \qquad [2]$$

Where:

Z = EA/c is the pile impedance
E = modulus of elasticity of pile material
A = cross sectional area of the pile
c = material wave speed

The wave assumes an opposite direction (a reflection) when it encounters soil resistance forces along the shaft or at the toe. These reflections travel upward along the shaft and arrive at the pile top at times that are related to their location along the shaft. The sensors near the pile top take measurements that translate what is happening to the traveling waves, and make it possible to estimate soil resistance and its distribution.

The data obtained in this fashion permits the computation of total soil resistance, which includes both static and viscous components. The dynamic component is computed as the product of the pile velocity times the damping factor (a soil parameter related to energy dissipation within soil). The static component is the total soil resistance minus the dynamic component). Dynamic load testing takes a further step in analyzing the data and computing static capacity and resistance distribution. Dynamic pile monitoring takes advantage of the fact that, for driven piles, it is possible to compute the energy delivered to the pile, compression stresses at the pile top and toe and tension stresses along the shaft. Pile damage can also be evaluated using this method.

The method has been successfully used to test most types of piles. In Canada, the method is typically used to verify the capacity of driven and to a lesser extend Cast in place concrete piles.

CAPWAP models the pile as a series of continuous segments. Each segment is of uniform cross-section but segments may be different from each other to accommodate non-uniform piles. A soil model similar to Smith's wave equation model is assumed that includes the total resistance and its distribution, damping constants and quake.

The CAPWAP results are based on the best possible match between computed pile top variables (i.e. the pile top force) and its measured equivalent.

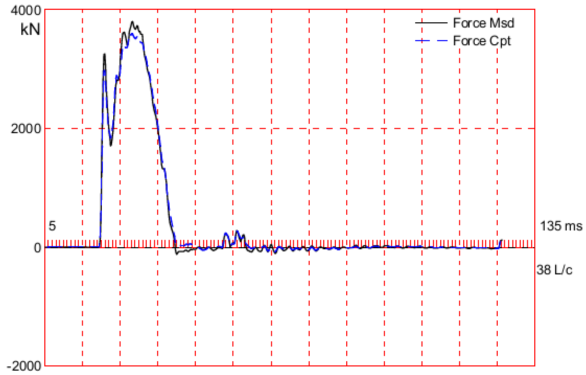

Figure 6. Measured and computed force profile of a pile

The traces of force and velocity measured in the field is matched with force and velocity computed by CAPWAP to provide the foundation design engineer with the following wave equation parameters:

- Applicable Case Method Estimates of capacity;
- Shaft resistance (magnitude and distribution);
- Toe resistance;
- Shaft and toe damping;
- Shaft and toe quake; and
- Simulated pile behavior under static analysis.

It is important to note that values obtained at site are approximate, particularly pile capacities. These preliminary estimates are highly dependent on pile type and geometry as well as other site conditions. As a result, more accurate analysis such as CAPWAP is used to obtain refined results. Figure 7 shows a typical CAPWAP output that is generated during an analysis of pile A3.

windy flats snc lavalin; Pile: A3 RESTRIKE — Test: 22-Nov-2014 10:28
junttan (323.9mm x 12.7mm x 16.50m); Blow: 96 — CAPWAP(R) 2014-1
Morton & Jagodich Inc — OP: VC

CAPWAP SUMMARI RESULTS

Total CAPWAP Capacity: 1746.5; along Shaft 1521.7; at Toe 224.8 kN

Soil Sgmnt No.	Dist. Below Gages m	Depth Below Grade m	Ru kN	Force in Pile kN	Sum of Ru kN	Unit Resist. (Depth) kN/m	Unit Resist. (Area) kPa
				1746.5			
1	3.0	1.5	124.2	1622.3	124.2	82.80	81.37
2	5.0	3.5	155.3	1467.0	279.5	77.65	76.31
3	7.0	5.5	186.3	1280.7	465.8	93.15	91.54
4	9.0	7.5	217.4	1063.3	683.2	108.70	106.82
5	11.0	9.5	248.5	814.8	931.7	124.25	122.11
6	13.0	11.5	279.5	535.3	1211.2	139.75	137.34
7	15.0	13.5	310.5	224.8	1521.7	155.25	152.57
Avg. Shaft			217.4			112.72	110.77
Toe			224.8				2728.25

Soil Model Parameters/Extensions		Shaft	Toe
Smith Damping Factor		0.26	0.67
Quake	(mm)	2.6	3.5
Case Damping Factor		0.78	0.30
Damping Type		Viscous	Sm+Visc
Unloading Quake	(% of loading quake)	30	57
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	4	
Resistance Gap (included in Toe Quake) (mm)			2.5
Soil Plug Weight	(kN)		0.902

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CAPWAP match quality   =    3.53       (Wave Up Match) ; RSA = 0
Observed: Final Set    =    15.6 mm;   Blow Count    =    64 b/m
Computed: Final Set    =    14.2 mm;   Blow Count    =    70 b/m
Transducer  F2(K807)   CAL: 148.9; RF: 1.00; F4(K768)  CAL: 150.9; RF: 1.00
            A1(K4634)  CAL:   365; RF: 1.00; A3(K4631) CAL:   323; RF: 1.00
max. Top Comp. Stress  =   218.9 MPa   (T=  21.5 ms, max= 1.054 x Top)
max. Comp. Stress      =   230.7 MPa   (Z=   3.0 m, T=  21.9 ms)
max. Tens. Stress      =  -14.37 MPa   (Z=   7.0 m, T=  43.9 ms)
max. Energy (EMX)      =    49.2 kJ;    max. Measured Top Displ. (DMX)=  20.1 mm
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Figure 7. Typical CAPWAP summary table of computed pile dynamic parameters

The PDIPLOT program will directly read data from PDA W0I or X0I files and present them in a graphical and /or tabular form. The program allows the presentation in a variety of different ways.

PDA presents graphically the measured/combined values of the following PDA quantities vs. depth:

- Maximum transferred energy (EMX);
- Penetration resistance (blow count, BLC);
- Maximum force (FMX);
- Applicable CASE method estimate (RX#);
- Maximum compressive stress (CSX); and
- Maximum computed stress at the pile toe (CSB).

A total of eighty three (83) tests were conducted at the site. Thirty three (33) of the tested piles were 323.9 mm outer diameter, twenty one (21) piles were 508 mm outer diameter, seventeen (17) piles were 406.4 mm outer diameter, and two (2) piles were 273.1 mm outer diameter. All driven piles were open ended with a wall thickness of 12.7 mm. A Junttan HHK-5A hydraulic hammer with the rated energy of 59 kJ, was used to install and test the piles at the site.

Six piles were tested for both end of drive and re-strike as these piles did not have enough capacity at the initial drive. All other piles were tested for end of drive except eleven piles which were tested for re-strike. The scope of the test program was to determine the pile capacity and to determine if the pile were damaged during driving.

5 RESULTS AND INTERPRETATION

A total of seventy six (76) CAPWAP analysis were performed on a representative hammer blow record from the PDA data at re-strike and end of drive (EOD). CAPWAP analysis are performed mainly to verify the applicable CASE Method Estimates, and to determine soil parameters and resistance distribution for evaluating the test results. Generally, the mobilized static resistance computed by CAPWAP showed an agreement with CASE Method Estimate (CMES) RX9 or RX8. More over Davisson offset method (Davisson, 1972) also was used along with the simulate load deflection curve to calculate the ultimate load bearing capacity of each piles.

Pile driving parameters such as transferred energy, driving stresses, and penetration resistance for the selected hammer blow records are presented in Table 5 and Table 6 respectively.

Table 5: Geotechnical Design Parameter based on CAPWAP analysis

	Pile mark	P1a	P1b	P1c	P2a	P2b
Avg. Shaft	kN	2302	1860	2498	1803	2424
Std. of shaft	kN	546	650	330	392	752
Avg. Toe	kN	1034	893	1047	594	1123
Std. of Toe	kN	500	365	178	394	526
Avg. EMX	kN-m	54	58	51	49	50
Std. of EMX	kN-m	18	4	1	17	14
Avg. CSX	MPa	265	248	220	224	214.5
Std. of CSX	MPa	49	21	3	33	43
Avg. CSB	MPa	164	120	130	121	126.25
Std. of CSB	MPa	52	38	14	20	41

Note: Std. stands for Standard deviation

Table 6: Geotechnical Design Parameter based on CAPWAP analysis

	Pile mark	P3	P4	P5	P6
Avg. Shaft	kN	2555	3928	1599	1599
Std. of shaft	kN	492	911	893	893
Avg. Toe	kN	1298	2704	1277	1277
Std. of Toe	kN	78	1115	1042	1042
Avg. EMX	kN-m	54	58	55	55
Std. of EMX	kN-m	15	2	5	5
Avg. CSX	MPa	221	242	350	350
Std. of CSX	MPa	42	10	28	28
Avg. CSB	MPa	113	119	209	209
Std. of CSB	MPa	20	28	35	35

Note: Std. stands for Standard deviation

Results of the dynamic soil properties are summarized in Table 7 and Table 8. These parameters are useful in analysing Very dense sandy soil found in Southern Alberta.

Table 7: Dynamic Soil Parameter Derived from CAPWAP analysis for very dense coarse grained soil

	Pile mark	P1a	P1b	P1c	P2a	P2b
Smith Damping (s/m)	Avg. Shaft	0.4	0.5	0.4	2.2	0.6
	Std. of shaft	0.2	0.3	0	3.5	0.4
	Avg. Toe	0.2	0.3	0.2	0.2	0.3
	Std. of Toe	0.1	0.3	0.1	0.1	0.5
Quake Value (mm)	Avg. Shaft	1.7	2.5	3.1	4.7	3.1
	Std. of shaft	1.1	1.9	2.1	2.1	2.3
	Avg. Toe	9	10.3	7.2	10.7	8.1
	Std. of Toe	2.3	5	0.5	7.9	4.7

Note: Std. stands for Standard deviation

Table 8: Dynamic Soil Parameter Derived from CAPWAP analysis for very dense coarse grained soil

		Pile Mark	P3	P4	P5	P6
Smith Damping	Avg. Shaft	s/m	0.7	1	0.7	0.5
	Std. of shaft	s/m	0.2	0.3	0.2	0.4
	Avg. Toe	s/m	0.2	0.3	0.2	0.9
	Std. of Toe	s/m	0.1	0.4	0.1	2.2
Quake Value	Avg. Shaft	mm	2.3	1	2.3	2.9
	Std. of shaft	mm	1.8	0.1	1.8	1.9
	Avg. Toe	mm	2.2	2.4	2.2	5.2
	Std. of Toe	mm	1.7	2.4	1.7	4.4

Note: Std. stands for Standard deviation

The majority of the tested pile's ultimate capacities were more than the required ultimate capacity. Measured compressive and tensile driving stresses were mainly within the acceptable limits for Grade 3 or higher steel throughout the test; however, high driving energies were measured in some locations during end of drive (EOD). All the tested piles met the ultimate axial capacity requirement.

6 CONCLUSION

Statistically speaking, CAPWAP results provides a reliable and rapid way to test and approve piles during construction.

As shown in Figure 8, most of the piles met the compressive capacity requirement stipulated in the design. However, as it stands right now, the design based on the empirical methods have proved to be expensive with the design parameters being conservative. As a design parameter optimization process, few high strain dynamic load testing during investigation phase of the project would have saved significant amount money on foundation construction.

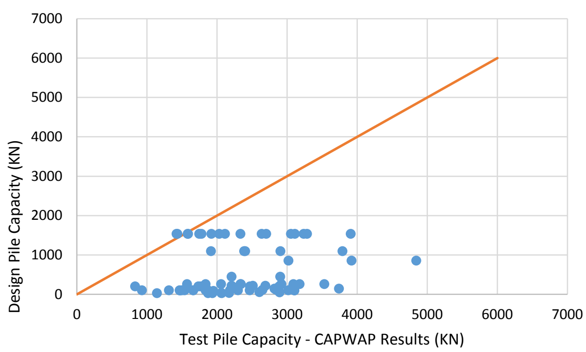

Figure 8. Variation of pile design capacity vs. CAPWAP capacity

Some of the piles did not meet the uplift capacity requirement of the piles, as such these piles were re-driven and were tested again to verify the uplift capacity requirements. At the end of re-drive, these piles were found to meet the capacity requirement.

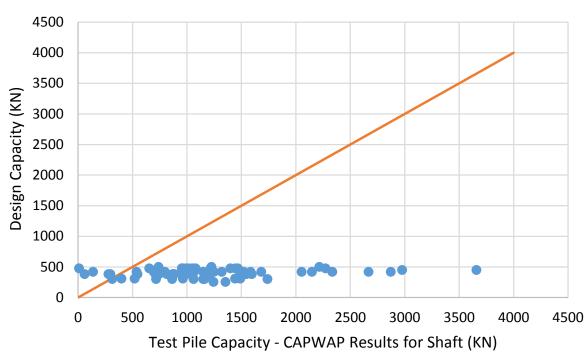

Figure 9. Variation of pile design shaft capacity vs. CAPWAP computed shaft capacity

Load settlement curves generated from CAPWAP analysis was normalized and used in checking the load deformation characteristics of the piles. The normalized load settlement curve for very dense sandy soil found in southern Alberta is provided in Figure 10 below.

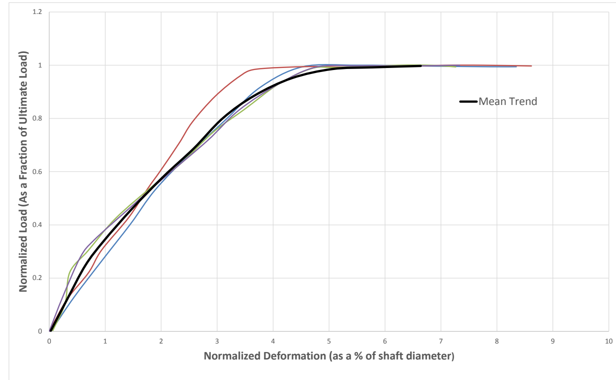

Figure 10. Load settlement characteristic curve for very dense soil found in southern Alberta

The characteristic curve provided in Figure 10 indicate that the piles reached their ultimate capacities around 5 percentage of their normalized deformation. Consequently, smaller piles would have smaller settlement tolerance while the larger diameter piles would have larger settlement tolerance. For structural purposes, the piles are assumed to have a deflection of 25 mm under SLS condition.

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