Driven Steel Piles in Clay Shale on Northeast Anthony Henday Drive in Edmonton

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ABSTRACT
The Northeast Anthony Henday Drive (NEAHD) project included the construction of 27 kilometers of six- and eight-lane divided roadway, nine interchanges, two road flyovers, eight rail bridges, and two bridges across the North Saskatchewan River, for a total of 47 bridge structures. The project was broken down into three smaller segments. The majority of the middle segment bridge abutments and bridge piers were supported on steel H-piles driven into the underlying clay shale bedrock. The QA program for the bridge pile installations included geotechnical monitoring of pile installations, including PDA/CAPWAP testing on approximately 10 percent of installed piles. This paper describes the results of geotechnical investigations carried out for the middle segment bridge structures, provides a description of the design and construction process, provides back-calculated pile design parameters based on construction monitoring results, and presents comparisons between pile design parameters that were adopted based on the geotechnical investigation results and the results of PDA/CAPWAP tests that were undertaken as part of quality assurance activities during bridge construction.

RÉSUMÉ
Le projet Nord-Est Anthony Henday Drive (NEAHD) comprenait la construction de 27 kilomètres de routes à chaussées séparées de six et huit voies, neuf échangeurs, deux survols routiers, huit huit ponts ferroviaires et deux ponts traversant la rivière Saskatchewan Nord, pour un total de 47 structures de ponts. Le projet a été divisé en trois segments. La majorité des culées et des piliers de pont du segment étaient soutenues sur des pieux d’acier en H. Les pieux d’acier étaient enfoncés dans le rocheux de schiste argileux. Le programme de qualités de construction pour les installations de pieux comprenait la surveillance géotechnique des installations et des essais PDA / CAPWAP sur environ 10% des pieux installées. Cet article décrit les résultats des études géotechniques réalisées pour les structures de ponts pour le segment du milieu. L’article décrit le processus de conception et de construction,fournit des paramètres de conception de pieux calculés en fonction des résultats de la surveillance de la construction, et présente des comparaisons entre les paramètres de conception des pieux adoptés en fonction des résultats de l'étude géotechnique et des résultats de PDA / CAPWAP qui ont été entrepris dans le cadre des activités d'assurance de la qualité pendant la construction du pont.

1 INTRODUCTION

Anthony Henday Drive (AHD) is the ring road highway around Edmonton, AB. The Northeast Anthony Henday Drive (NEAHD) was the final leg to be completed and opened to traffic by Alberta Transportation in October 2016. The NEAHD was a P3 project involving the construction of 27 km of 8-lane divided roadway, 9 interchanges, 2 road flyovers, 8 rail crossings and two bridges crossing the North Saskatchewan River for a total of 47 bridge structures. The location of the NEAHD is shown in Figure 1.

Due to the large size of this project and the aggressive construction schedule, the project was broken down into three segments, such that geotechnical investigations, foundation designs and construction monitoring could be carried out concurrently for the entire project by separate consultant teams. The middle segment of the project included 23 new bridge structures at locations along Yellowhead Trail (from Sherwood Drive to the North Saskatchewan River) and along NEAHD (from Petroleum Way to Hayter Drive). Eighteen of the middle segment bridge abutments and piers were supported on steel H-piles driven into the underlying bedrock. The remaining five bridges were located within historic coal mine areas and therefore they are excluded from this paper. Foundations for bridges over abandoned coal mines were discussed by Soliman and Walter (2016). The middle segment of the project was further broken down into seven sites as described below. The site locations are shown in Figure 2.

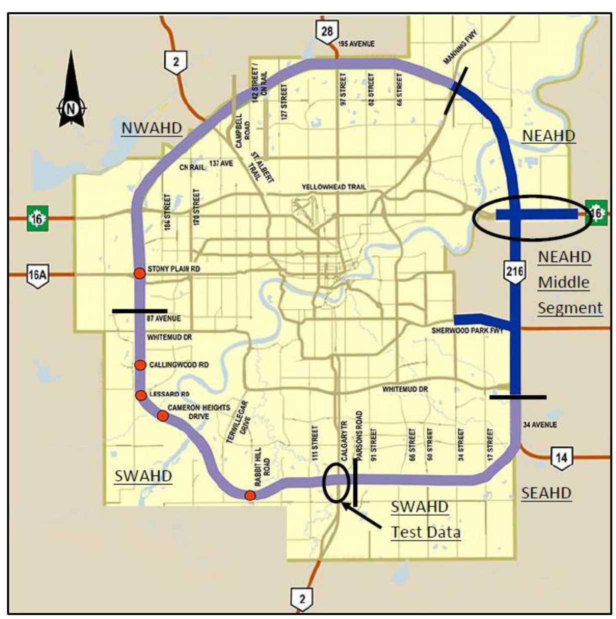

Figure 1. Map of Edmonton highlighting AHD ring road and project sites (Alberta Transportation)

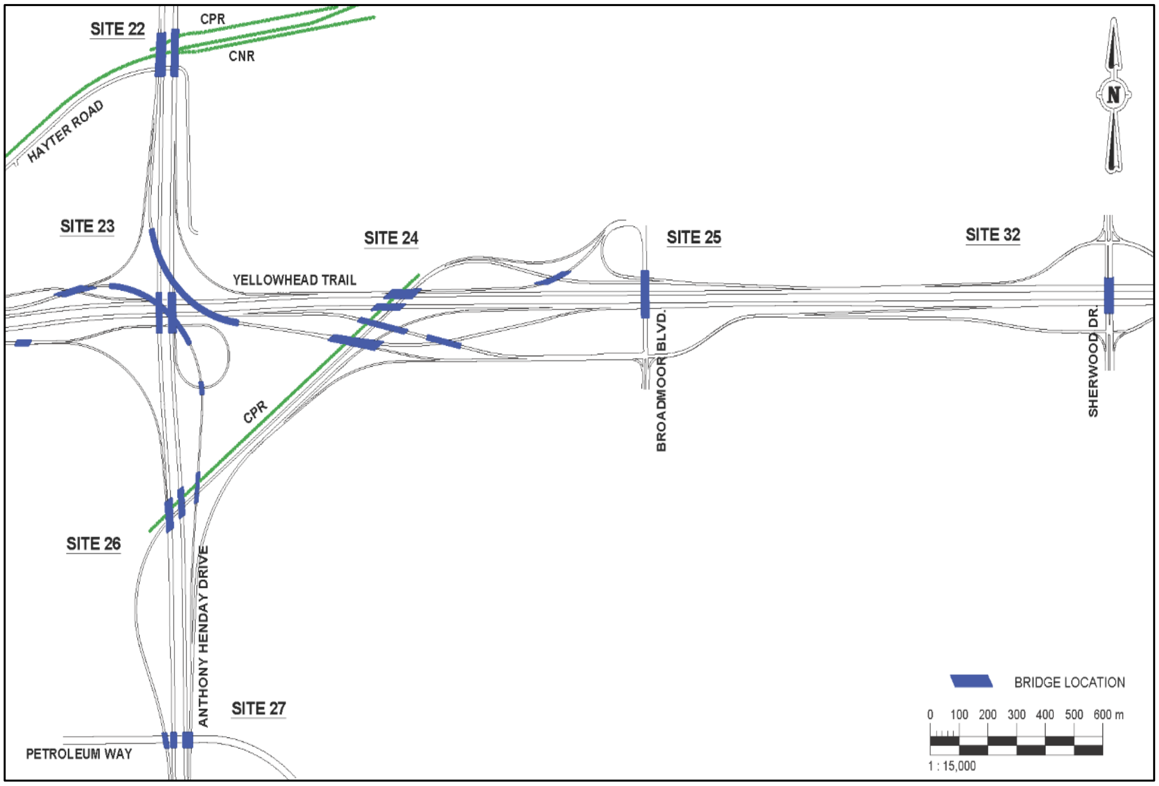

Figure 2. Middle segment bridge structure locations.

- Site 22: The north end of the middle segment where NEAHD crossed over Hayter Road, Canadian National Railway (CNR) and Canadian Pacific Railway (CPR). Site 22 included two bridge structures.
- Site 23: The intersection of Yellowhead Trail (YHD) and AHD area. Site 23 includes a total of seven new bridge structures, but two of them were excluded from this paper as they are located within the coal mine area.
- Site 24: East of Site 23 where AHD ramps go over YHD and CPR. Site 24 consisted of five bridge structures.
- Site 25: East of Site 24 where Broadmoor Boulevard crossed over YHD. Site 25 consisted of two bridge structures.
- Site 26: The site included three bridge structures located within the coal mine area, which were excluded from this paper.
- Site 27: The south end of the middle segment along NEAHD where it crossed over Petroleum Way. Site 27 consisted of three bridge structures.
- Site 32: The east end of the middle segment where Sherwood Drive crossed over YHD. Site 32 consisted of one bridge structure.

Driven steel HP360x132 H-piles were used for foundations of all bridge structures, excluding the coal mine bridge structures. The design unfactored geotechnical resistance per single pile was in the order of 3000 kN. The design of these piles was carried out using conventional pile design methods (CFEM 2006), and was based on achieving resistance by skin friction and end-bearing.

During the pile installation, PDA (Pile Driving Analyzer) testing and CAPWAP (CAse Pile Wave Analysis Program) were carried out as part of a quality assurance (QA) program, in order to confirm that the installed piles achieved their design capacities. Most of the H-piles attained their required capacities during initial driving, with a small percentage requiring restrike testing to confirm that the required capacities were reached.

This paper presents a summary of the geotechnical investigation carried out at the NEAHD site, subsurface soil and bedrock conditions, the PDA/CAPWAP test data that were collected during QA activities, and discussion of the test results. PDA/CAPWAP test data collected during pile installations for the Southwest Anthony Henday Drive (SWAHD) was also reviewed for comparison with data from NEAHD.

2 GENERAL SITE GEOLOGY

The generalized geology in the project area described in Kathol and McPherson (1975) comprises of thin glacio-lacustrine deposits consisting mainly of clay and silt over glacial clay till, followed by pre-glacial sand and gravel of the Empress Formation, over bedrock. Within the glacial till, are zones which are interspersed with water-bearing sand and gravel, and zones possibly containing cobbles and boulders. Within the area, a preglacial buried valley incised into bedrock and infilled with fluvial deposits of the Empress Formation underlies the till. The thalweg (deepest part of the channel) is located in the area of the NEAHD project site. The depth to bedrock is about 10 to 30m in the area and is expected to vary depending on the specific location relative to the pre-glacial channel that extends through the study area. The near-surface bedrock in the study area is the Horseshoe Canyon Formation of the late Cretaceous age. The upper bedrock consists mainly of claystone (locally referred to as clay shale) interbedded with fine-grained sandstone and siltstone. Coal seams and bentonitic beds are common throughout the formation. According to the geological maps and profiles provided by Kathol and McPherson (1975), the bedrock surface dips gently in the northwest direction.

3 SUBSURFACE CONDITIONS

A geotechnical investigation of the middle segment of NEAHD was carried out during the spring and summer of 2012. The investigation included drilling of 85 augered boreholes and 18 wet-rotary holes, in-situ soil testing, groundwater monitoring, and laboratory testing on collected samples. As part of in-situ soil testing, Standard Penetration Tests (SPT) were conducted in the boreholes at approximately 1.5 to 3 m depth intervals to evaluate the consistency and/or relative density of the soil strata. Disturbed samples were obtained for various lab-based testing and detailed examination in the laboratory. Thin-walled Shelby tube samples of cohesive soils were collected for strength tests in the laboratory.

Throughout the project site, various thickness of fill, typically less than 1 m in thickness was observed at surface. At Site 23, preplaced fills up to 15 m thick were present at the locations of some of the ramps. The existing fill varied from medium plastic clay fill to sandy gravel fill.

Generalized soil stratigraphy observed at each site is described below. SPT 'N' values (number of blows per 300 mm of penetration) distribution of each site is shown in Figures 3(A) to 8(A). Blow counts higher than 100 were plotted as 100 on the figures.

- Site 22: Approximately 1.5 m thick lacustrine clay was present below the surficial fill material (about 1 m thick). Below the clay, about 2 m thick clay till was present overlying Empress sand. The bedrock was present at about 10 m below the ground surface. Groundwater level was approximately 5 m below ground surface.
- Site 23: Preplaced fills up to 15 m thick were present at the locations of some of the ramps. The existing fill varied from medium plastic clay fill to sandy gravel fill. Clay and clay till (up to 3 m) was observed below the fill, followed by outwash sand, followed by clay till that extended to bedrock. About 15 to 20 m of native soils were present over bedrock. Groundwater level was approximately 2 to 5 m below ground surface.
- Site 24: Roadway embankment fill up to 8-10 m in thickness was observed at surface and consisted primarily of clay fills. Clay till was present below the fill and extended to about 15 m depth, followed by Empress sand to 20 to 25 m, followed by sandstone bedrock. Groundwater level was approximately 2 m below ground surface.
- Site 25: Thin layers of topsoil and clay fill were observed at surface, followed by clay till to about 8 m depth, followed by Empress sand to approximately 15 to 20 m depth, over bedrock. Groundwater level was approximately 5 m below ground surface.
- Site 27: Road embankment clay fills, approximately 8 m in thickness, were encountered at surface, followed by clay till that extended to bedrock at about 25 m depth. Groundwater level was approximately 5 m below ground surface.
- Site 32: Below the surficial fill and topsoil, 15 m of clay till was present. About 5 to 10 m thick Empress sand deposit overlying the bedrock was observed beneath the clay till. Groundwater level was approximately 1 to 2 below ground surface.

Relatively thin layers of lacustrine clay were observed above the clay till at some locations. The lacustrine clay was typically silty, stiff and medium to high plastic. The clay till was typically a silty, sandy and low to medium plastic clay matrix, and it contained gravel inclusions, coal fragments and rust stains. The moisture contents of the clay till typically varied between 15 and 25 percent, which was near the plastic limit of the till. SPT 'N' values observed in clay till were typically in the 10 to 30 range indicating consistencies ranging from firm to hard. Various thickness and composition of sand deposits and rafted bedrock were found within the clay till. Although not usually retrieved by an auger, cobbles and boulders are commonly present within the clay till in the general area.

The outwash sand observed above the till at Site 23 was fine to coarse grained, contained gravel sizes, and was generally compact (N values between 20 and 30) with some dense zones (N > 30). The Empress sand deposits present beneath the clay till were fine to coarse grained sand, and contained various amount of silt, gravel sizes and random clayey zones. SPT 'N' values in the Empress Sand was generally dense to very dense, with N values typically above 30.

Clay shale bedrock with sandstone interbeds was encountered at depths varying between 10 and 30 m below the ground surface at the time of investigation. The clay shale was typically silty, medium to high plastic and contained sandstone interbeds, lenses and pockets. The sandstone was fine grained, silty and blueish grey color. Varying sizes of coal and bentonitic seams were present throughout the bedrock. The unconfined compression test results show that the upper bedrock (upper 10 m) had an average compressive strength of 430 kPa and lower bedrock (10 to 30 m depth from the top of bedrock) had an average of 660 kPa. Based on the unconfined compressive strength of the observed bedrock, it is considered very soft rock or hard soil-like material on rock strength scale (NRCS 2012).

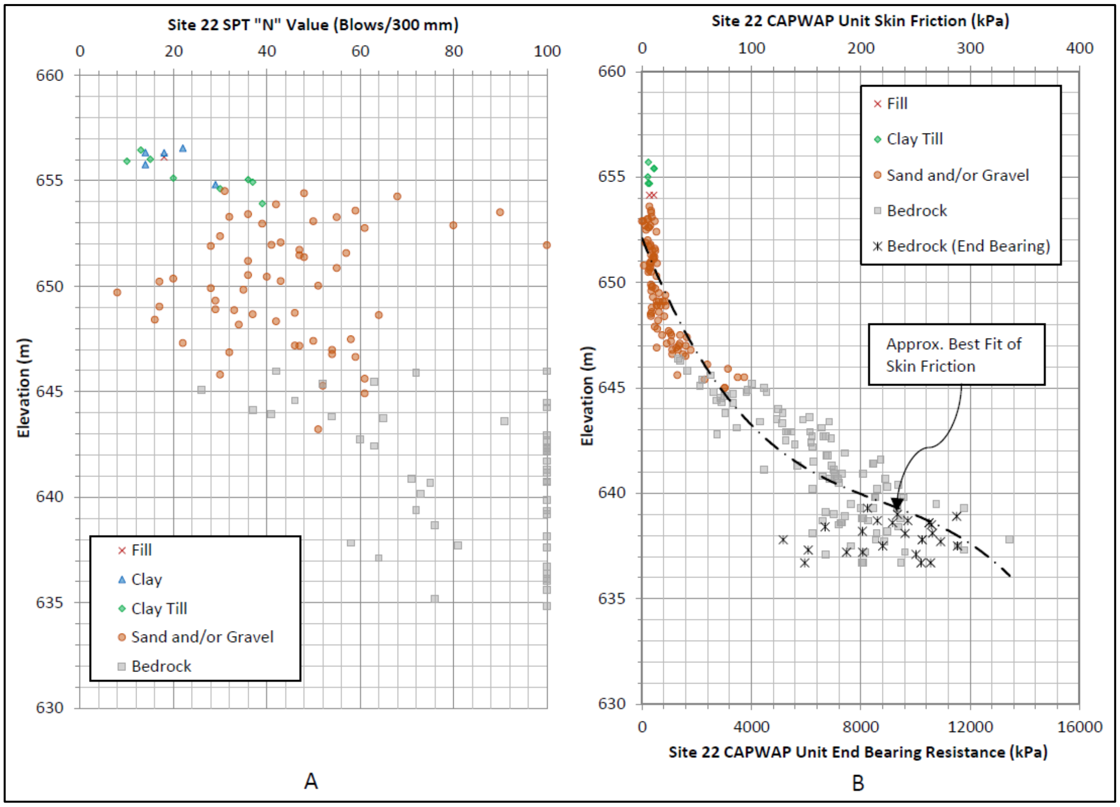

Figure 3. A) Site 22 variation of SPT ‘N’ values with elevation
B) Site 22 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

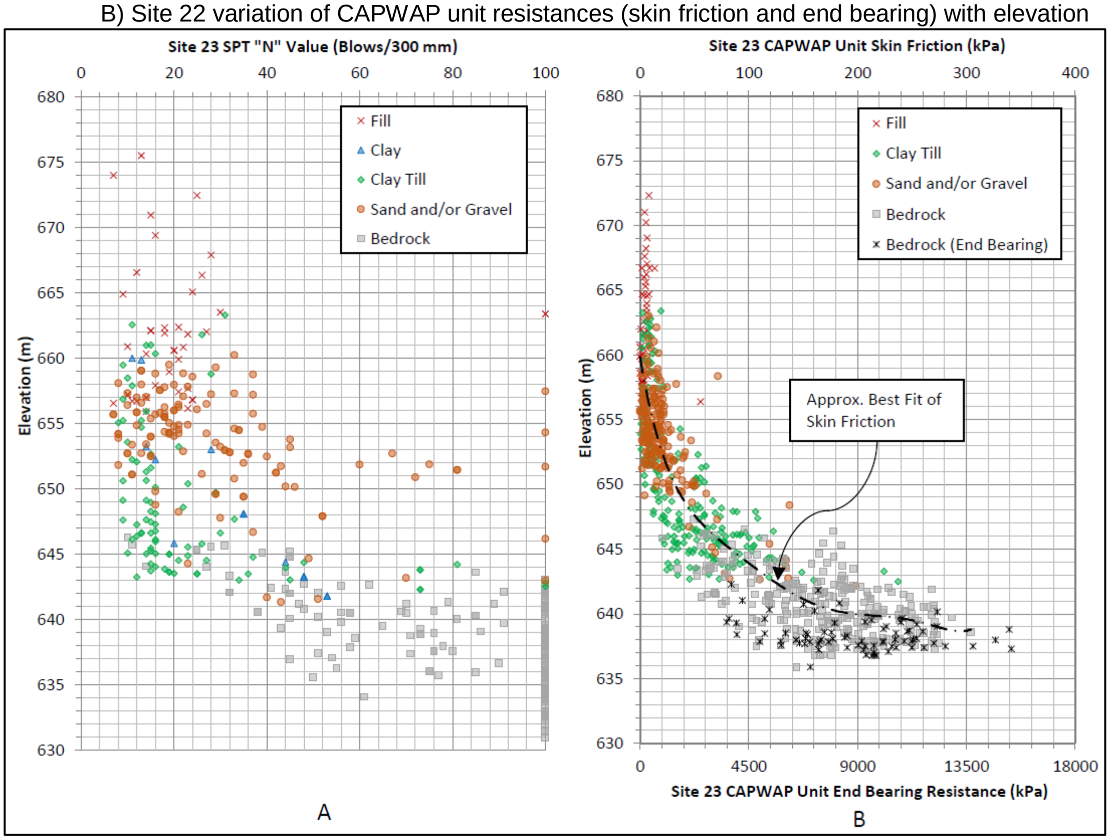

Figure 4. A) Site 23 variation of SPT ‘N’ values with elevation
B) Site 23 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

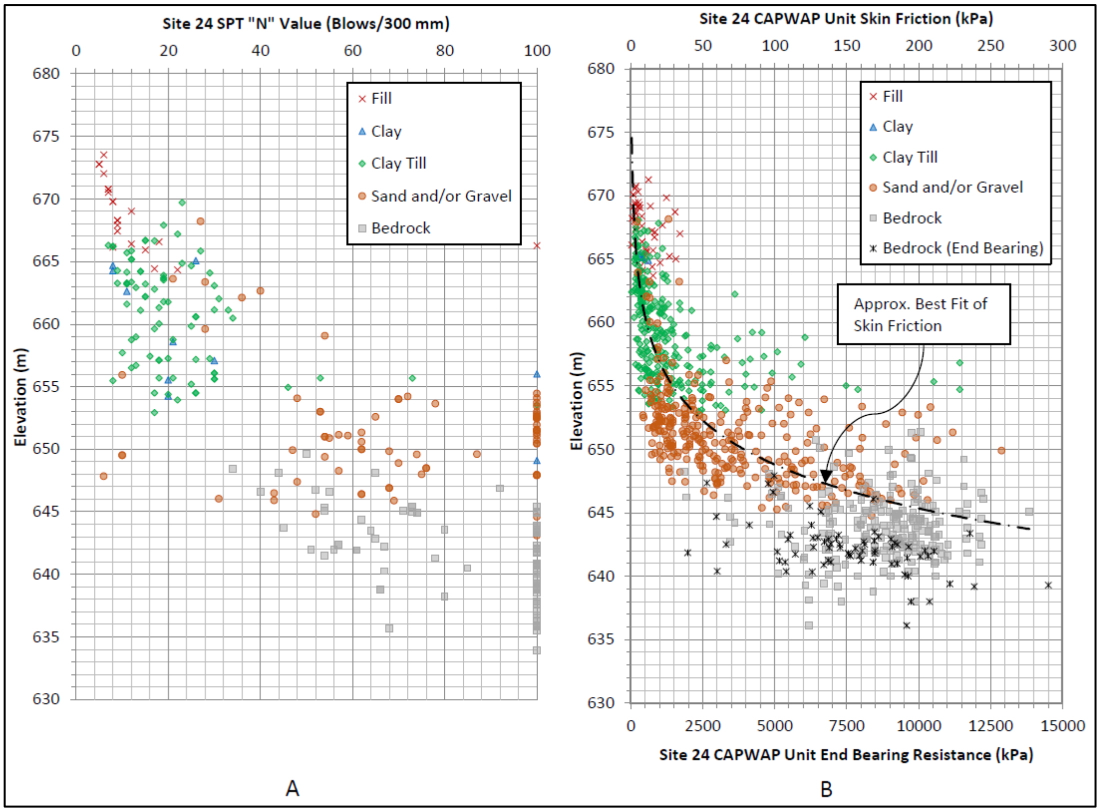

Figure 5. A) Site 24 variation of SPT 'N' values with elevation
B) Site 24 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

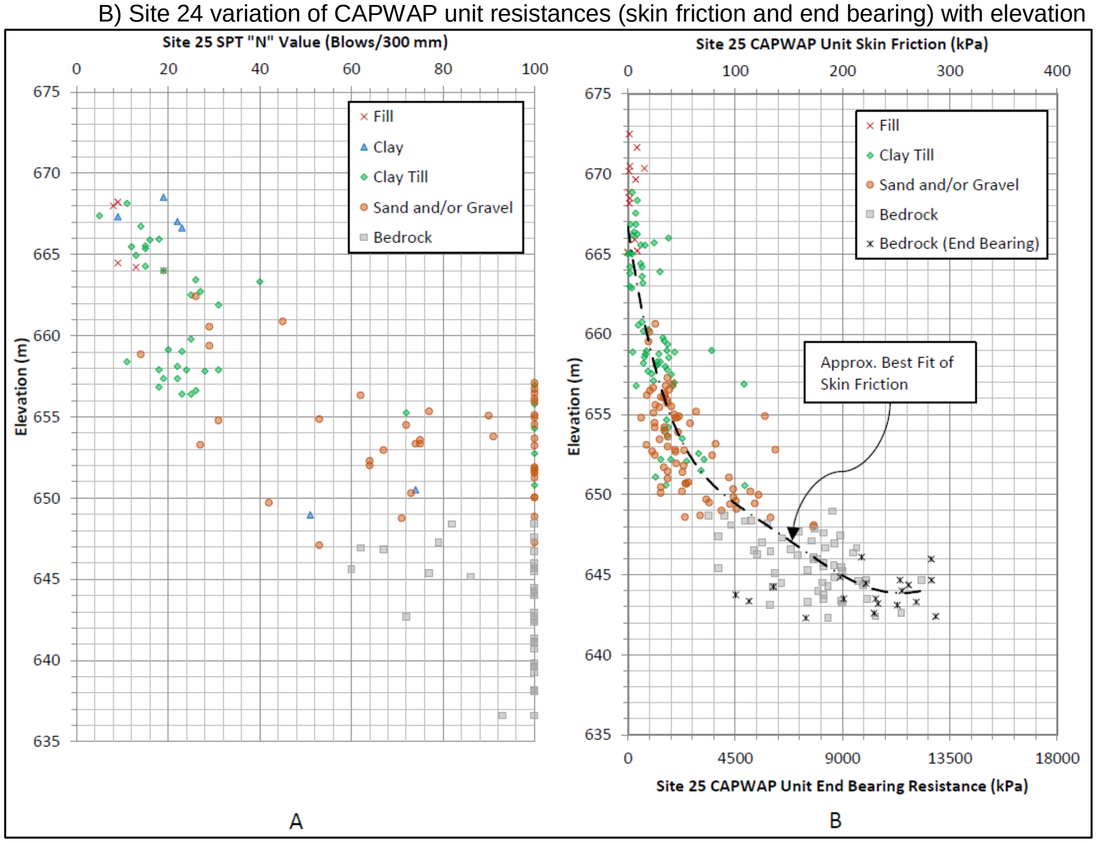

Figure 6. A) Site 25 variation of SPT 'N' values with elevation
B) Site 25 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

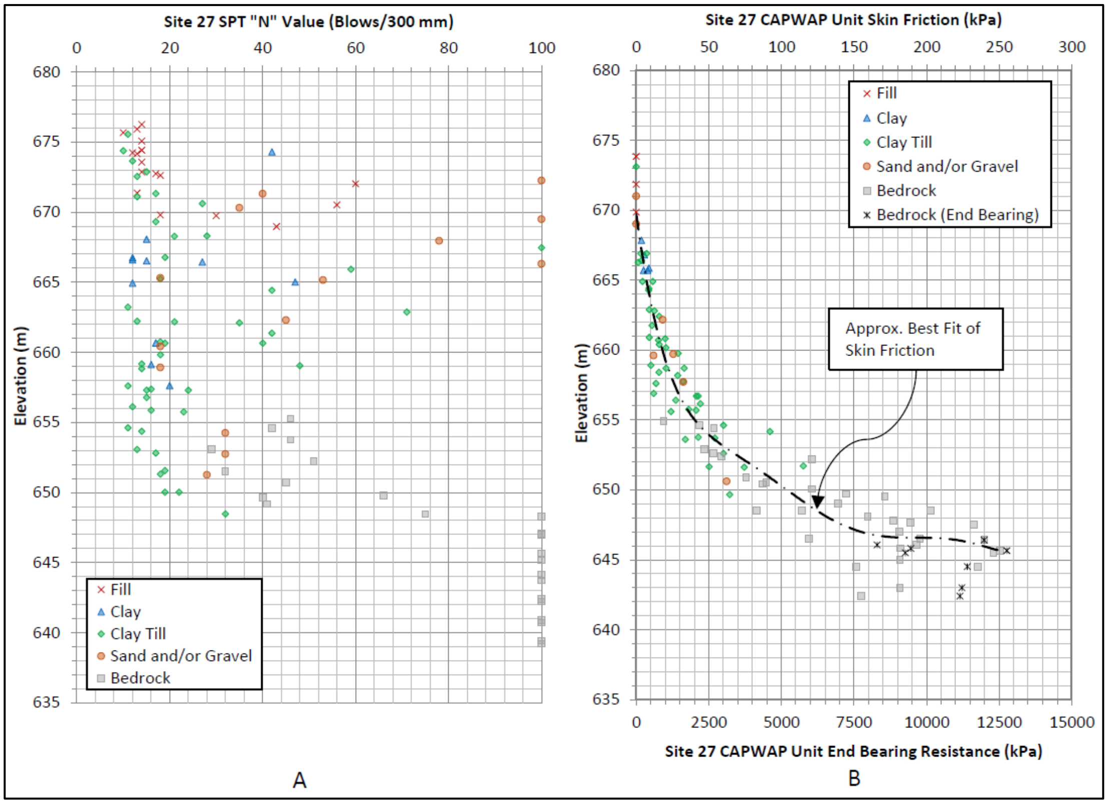

Figure 7. A) Site 27 variation of SPT 'N' values with elevation
B) Site 27 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

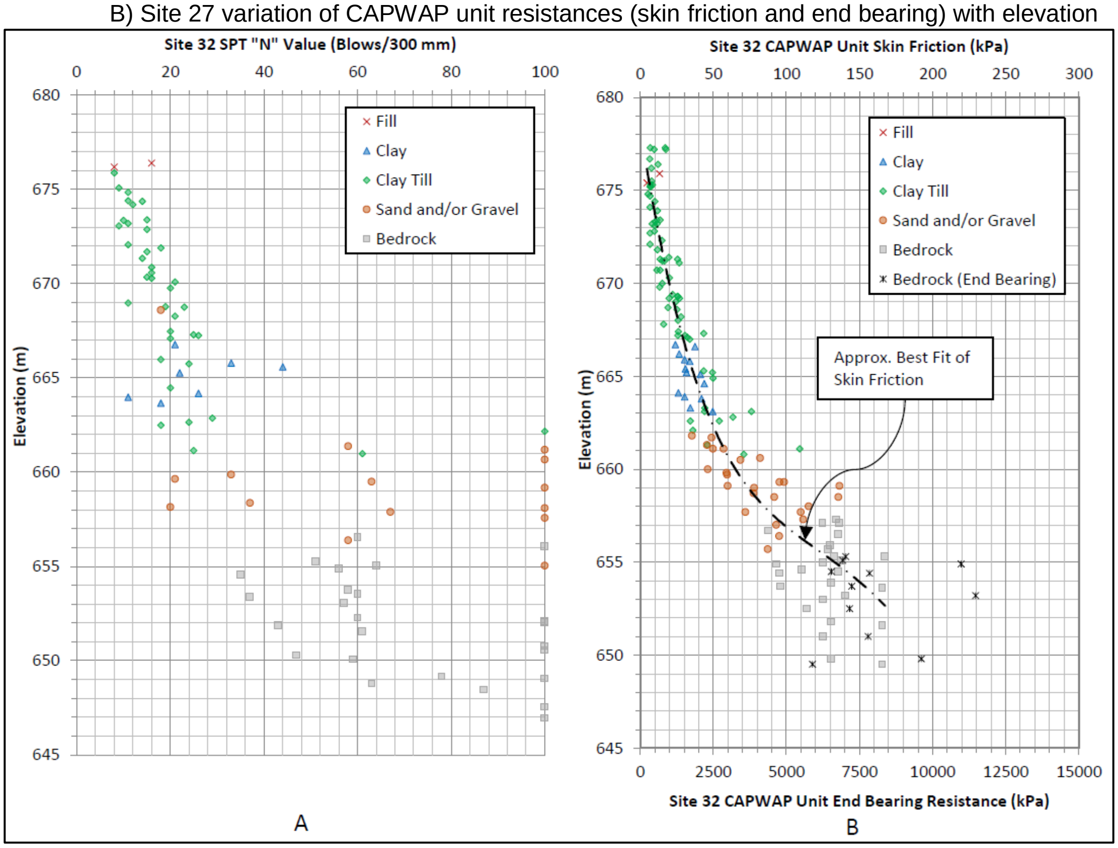

Figure 8. A) Site 32 variation of SPT 'N' values with elevation
B) Site 32 variation of CAPWAP unit resistances (skin friction and end bearing) with elevation

4 PILE INSTALLATION AND MONITORING

Berminghammer B5505 and APE D50-42 diesel hammers affixed to cranes were utilized for pile installation. At the contractor's request and wherever possible, the piles were driven to their respective design ultimate geotechnical resistances during the initial driving phase, meaning no reliance was made on soil set-up effects following initial pile installation. This was to minimize the number of mobilizations to the numerous individual piling locations located within or adjacent to active roadways. The lower segment of every pile was fitted with a driving shoe to limit the potential of damage to the pile toe during driving. In total, 1789 piles were driven for bridge structures at Sites 22, 23, 24, 25, 27, and 32 of the NEAHD project. Majority of the piles extended between 20 and 35 m below existed ground surface, with a total embedment length of about 40,000 m.

4.1 Pile Driving Criteria

The first phase of the QA program consisted of determination of pile driving termination criteria using GRLWEAP (2010) software combined with on-site pile monitoring for the installation of each pile. GRLWEAP termination charts were created to establish the driving energy/driving resistance/ultimate geotechnical resistance relationships for every specific bridge site considering the soil conditions/parameters at each bridge location and the specified installation hammer. Pile monitoring consisted of recording observed hammer driving energy (hammer blows per minute) and driving resistance (blows per 0.25 m of pile penetration) for the entirety of each pile penetration from surface to termination depth, measurements of verticality (plumb-ness) and pile splice locations, and any observations pertaining to potential pile damage. Pile driving was terminated once the observed driving energy and driving resistance combinations for 3 successive 0.25 m pile penetration intervals, when compared to the GRLWEAP termination charts, indicated the ultimate geotechnical resistance requirements had been achieved. The piles typically penetrated 4 to 6 m into bedrock at termination of driving.

4.2 PDA/CAPWAP Testing and Analysis

The second phase of the QA program was the execution of PDA (Pile Driving Analyzer) testing and CAPWAP (CAse Pile Wave Analysis Program) (2012) analysis on a minimum of 10 percent of piles belonging to each individual bridge element (abutments, piers or straddles). In total, 226 End of Initial Drive (EOID) PDA/CAPWAP tests/analyses were conducted.

The CAPWAP computed unit skin friction and end bearing resistances for each site are shown in Figures 3(B) to 8(B).

Test piles for each bridge element were identified prior to pile driving and were chosen such that the 10 percent minimum was satisfied, as well as to provide a spatial distribution of test data within a given bridge element footprint (i.e. piles chosen as far apart as possible). The designated test piles were then prioritized in the driving sequence so CAPWAP analysis could be undertaken in a timely fashion. The results of the CAPWAP analyses were utilized to calibrate the GRLWEAP charts that were used to estimate ultimate geotechnical resistances for piles that were not PDA tested.

4.3 CAPWAP Results

The unit skin frictions and end bearing resistances calculated from the CAPWAP analyses are presented on Figures 3(B) to 8(B), for Sites 22, 23, 24, 25, 27, and 32.

The back calculated unit skin frictions generally increased with depth through the overburden soil layers, followed by a significantly larger increase near/within bedrock. Considering all 226 PDA/CAPWAP tests and analyses, the resistance distribution at EOID was typically 60% to 70% shaft (30% to 40% end bearing). The shaft resistance distribution between overburden and bedrock soil layers was determined to range from 45% to 90% attributable to bedrock depending on the site, with the lower bound occurring at shallower bedrock depths and the higher bound where depth to bedrock was largest. Overall, the total percentage of resistance developed within the bedrock varied from 65% to 93%.

5 DISCUSSION

5.1 Total and Effective Stress Pile Design Methods

The geotechnical resistance of driven piles is commonly calculated in local practice as a combination of both skin friction along the pile shaft and end-bearing resistance at the toe. Total stress and effective stress methods for pile design were summarized in CFEM (2006). The unit shaft and end bearing resistances in cohesive materials were determined using the total stress method presented as Equations 1 and 2.

$$q_s = \alpha\, S_u \qquad [1]$$

where

q_s = unit shaft resistance

α = adhesion factor proportion to the undrained shear strength of the soil (CFEM curve re-created in Figure 9)

S_u = undrained shear strength of soil

$$q_t = N_t\, S_u \qquad [2]$$

where

q_t = unit end-bearing resistance

N_t = bearing capacity factor

= 9 (for pile diameter smaller than 0.5 m)

The unit shaft and end bearing resistances for cohesive and cohesionless soils were determined using the effective stress method provided in CFEM (2006), which are presented as Equations 3 and 4 below.

$$q_s = \beta\, \sigma'_v \qquad [3]$$

where

β = combined shaft resistance factor

σ'_v = vertical effective stress adjacent to the pile at a given depth

$$q_t = N_t \sigma'_t \quad [4]$$

where

N_t = toe bearing capacity factor

σ'_t = vertical effective stress at the pile toe

5.2 Back Analysis of Design Parameters Using CAPWAP Results

The PDA/CAPWAP derived shaft and the end-bearing resistance of the piles at the site were compared to resistences that were predicted using the total and effective stress methods as per CFEM (2006). The skin friction resistance was calculated using the surface area of the box bounded by the flanges of the H-pile. The end bearing resistance was based on a fully plugged base; i.e. base area equals the flange width x web length. The consistency and relative density of soils were estimated based on SPT data and unconfined compressive tests. For cohesive soils, the SPT 'N' values were multiplied by the locally used imperial factor of 5.5 to estimate the undrained shear strength. The estimated undrained shear strengths of the clay till and the bedrock were used to predict the unit resistance of the piles.

The adhesion factor (α), combined shaft resistance factor (β) and bearing capacity factor (N_t) were back calculated to establish the correlations for each site. The calculations for soil strength parameters and back calculated side and end bearing resistances were based on the mean and mean ± standard deviation for each parameter. The results are presented on Tables 1 to 4, and shown on Figures 9 to 11. The tables and figures also show the recommended values for α, β and N_t using the total stress method, and β and N_t using the effective stress method, according to CFEM (2006).

Table 1. Summary of back calculated and CFEM (2006) adhesion factors (α) using total stress method

Site	Soil Type	Range of Back Calculated α	α from CFEM (2006)
22	Bedrock	0.47 – 0.50	0.27 – 0.32
23	Clay till	0.31 – 0.53	0.39 – 0.71
	Bedrock	0.50 – 0.53	0.27 – 0.33
24	Clay till	0.09 – 0.33	0.40 – 0.62
	Bedrock	0.48 – 0.51	0.27 – 0.30
25	Clay till	0.17 – 0.30	0.39 – 0.56
	Bedrock	0.37 – 0.45	0.27 – 0.28
27	Clay till	0.18 – 0.29	0.37 – 0.67
	Bedrock	0.45 – 0.62	0.30 – 0.34
32	Clay till	0.15 – 0.30	0.43 – 0.63
	Bedrock	0.41 – 0.44	0.28 – 0.32

The back calculated adhesion factors (α) values (Table 1 and Figure 9) generally varied between 0.1 and 0.3 for clay till and were lower than the values recommended by CFEM (2006). For bedrock, the back calculated α ranged between 0.35 and 0.62. The N_t values (Table 2) in bedrock generally varied between 21 and 25, which is much higher than 9 which is typically used in design, assuming soil-like bedrock.

Table 2. Summary of back calculated and CFEM (2006) bearing capacity factors (N_t) using total stress method

Site	Soil Type	Range of Back Calculated N_t	N_t from CFEM (2006)
22	Bedrock	21 – 25	9
23	Bedrock	24 – 25	9
24	Bedrock	21 – 23	9
25	Bedrock	24 – 26	9
27	Bedrock	26 - 30	9
32	Bedrock	23 – 26	9

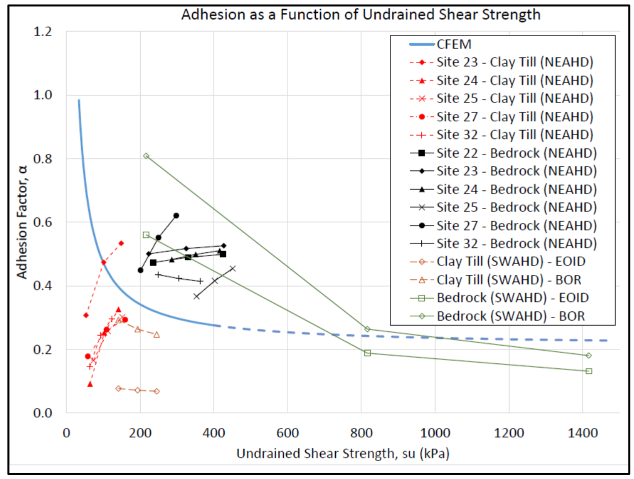

Figure 9. "Adhesion factor as a function of undrained shear strength" from CFEM (2006) including NEAHD and SWAHD data

Table 3. Summary of back calculated and CFEM (2006) combined shaft resistance factors (β) using effective stress method

Site	Soil Type	Range of Back Calculated β	β from CFEM (2006)
22	Clay till	0.1 – 0.22	0.25 – 0.32
	Sand	0.05 – 0.26	0.8 -1.2
	Bedrock	0.2 – 1	0.25 – 0.32
23	Clay till	0.06 – 0.7	0.25 – 0.32
	Sand	0.6 – 0.4	0.8 -1.2
	Bedrock	0.1 – 1	0.25 – 0.32
24	Clay till	0.05 – 0.35	0.25 – 0.32
	Sand	0.05 – 0.4	0.8 -1.2
	Bedrock	0.19 – 0.68	0.25 – 0.32
25	Clay till	0.06 – 0.51	0.25 – 0.32
	Sand	0.05 – 0.35	0.8 -1.2
	Bedrock	0.25 – 0.67	0.25 – 0.32
27	Clay till	0.05 – 0.25	0.25 – 0.32
	Bedrock	0.18 – 0.61	0.25 – 0.32
32	Clay till	0.12 – 0.63	0.25 – 0.32
	Sand	0.19 – 0.48	0.8 -1.2
	Bedrock	0.36 – 0.66	0.25 – 0.32

For calculation of combined shaft resistance factors (β), simplified soil stratigraphy for each bridge site and corresponding CAPWAP data were considered. The estimated β for each soil unit are provided in Table 3 and Figures 10 and 11. The back calculated β for both clay till and sand were lower than the values given in CFEM (2006) for comparable soils. For bedrock, the calculated β was higher than the value of 0.25 to 0.32 recommended for cohesive soils. In bedrock, the back calculated N_t varied between 8 and 50, and was typically higher than the range provided in CFEM (2006) for cohesive soils.

Table 4. Summary of back calculated bearing capacity factors (N_t) using effective stress method

Site	Soil Type	Range of Back Calculated N_t	Nt from CFEM (2006)
22	Bedrock	21 – 43	3 - 10
23	Bedrock	13 – 50	3 – 10
24	Bedrock	8 – 31	3 – 10
25	Bedrock	13 – 39	3 – 10
27	Bedrock	26 – 38	3 – 10
32	Bedrock	19 – 31	3 - 10

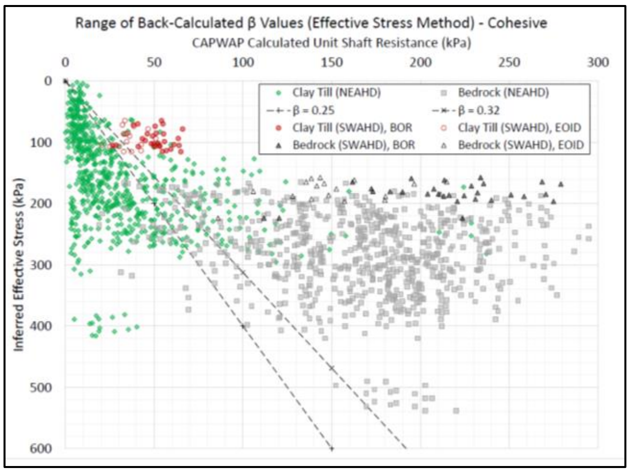

Figure 10. Back-calculated combined shaft resistance factors for cohesive soils from NEAHD and SWAHD data

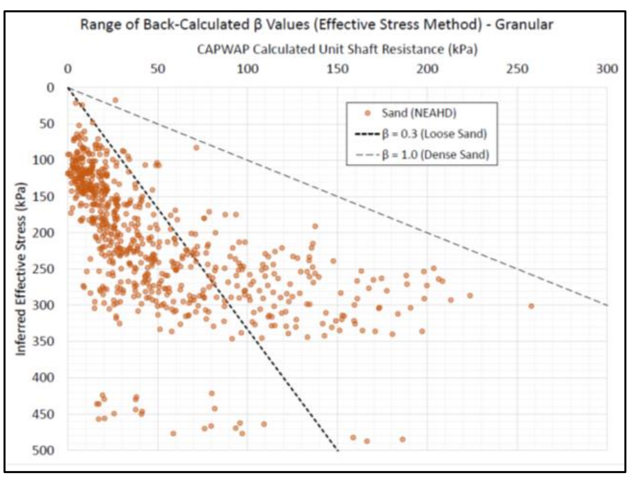

Figure 11. Back-calculated combined shaft resistance factors for granular soils from NEAHD data

The skin friction parameters in overburden soils back calculated from the CAPWAP analysis for NEAHD were typically lower than values provided in the CFEM (2006). The following may have contributed to the observed low skin friction resistances.

1. Use of a driving shoe and movement of the pile head during driving to maintain plumb-ness (crane mounted piling leads, not fixed at ground surface) can lead to a gap between the pile and soil in the upper soil strata.
2. Partial plugging in upper portion of H-pile and driving as a low displacement pile could lead to lower skin frictions.
3. The PDA testing and CAPWAP analyses were conducted under initial driving conditions, additional unit shaft resistance could potentially develop through soil set-up effects.

Relatively high skin friction resistances were observed in bedrock and at depth, which could be due to the development of a full soil plug.

5.3 Comparison of NEAHD and SWAHD Results

The results from a previous geotechnical investigation and QA program at a number of the South West Anthony Henday Drive (SWAHD) bridge structures were assessed in a similar manner to that described in the previous sections of this paper. Correlations between the CAPWAP data and soil conditions were reported by Chesham (2005). The area of interest included in Chesham (2005) is at the Calgary Trail Interchange, which included 4 new bridge structures. The project location is shown in Figure 1.

Generally, clay fill below topsoil was present at the surface of the SWAHD area. The thickness of the fill varied from 0 to 2.5 m throughout the site. Lacustrine clay was encountered below fill soils and extended 5 to 8 m below the ground surface at the time of investigation. The lacustrine clay was firm to stiff (mean SPT 'N' of 13.3 blows per 300 mm) and high plastic. Stiff to hard clay till (mean SPT 'N' of 35.1 blows per 300 mm) was underlying the clay. The clay till was silty, sandy, medium plastic and contained various sizes of sand deposits, coal fragments and rafted bedrock. Bedrock comprised of clay shale and sandstone was present beneath the clay till. The clay shale had an average undrained shear strength of 1560 kPa and the sandstone had an average of 1820 kPa. The groundwater table was present at 1.5 to 4 m below ground surface.

Driven steel H-piles HP310x94 embedded in the bedrock were used to support the bridge structures located at the Calgary Trail interchange.

As part of the QA program, 28 piles were subjected to PDA/CAPWAP testing for both EOID and BOR conditions. pile resistance generally increased with time due to set-up. Pile resistances calculated at BOR were typically 20 to 40 percent greater than at EOID. Chesham (2005) did not present estimates for adhesion factors and bearing capacity factors using the total stress method, however for the purpose of comparing to NEAHD data these values were calculated from the SPT 'N' and CAPWAP data presented in Chesham (2005). The interpreted values for adhesion factors (α) and bearing capacity factors (N_t) are presented in Table 5, and are plotted in Figure 9.

Table 5. Interpreted total stress parameters derived from SPT 'N' and CAPWAP data from Chesham (2005).

Soil Type	Range of Back Calculated α	Range of Back Calculated N_t
Clay Till (EOID)	0.07 – 0.08	-
Clay Till (BOR)	0.25 – 0.29	-
Bedrock (EOID)	0.13 – 0.56	7 – 34
Bedrock (BOR)	0.18 – 0.81	8 - 40

Using the computed BOR data, Chesham (2005) back-analyzed pile design parameters for the effective stress method (Equations 3 and 4). The SWAHD back calculated values for the shaft resistance factors (β) and bearing capacity factors (N_t) are summarized in Table 6 below. The back-calculated shaft resistance factors are plotted in Figure 10 for cohesive soils only. Granular soils were not prevalent on the SWAHD project.

Table 6. Effective stress parameters derived from CAPWAP back analysis at BOR. (Chesham 2005).

Soil Type	Range or Back Calculated β	Range of Back Calculated N_t
Clay Till (EOID)	0.29 – 0.50	-
Clay Till (BOR)	0.41 – 0.68	-
Bedrock (EOID)	0.66 – 1.08	39 - 53
Bedrock (BOR)	0.93 – 1.45	34 - 51

Similar to the NEAHD project data, back calculated values for adhesion factor (α) using the total stress method were consistently lower than those recommended by CFEM (2006). However the back calculated value for bearing capacity factor (Nt) using total stress method, as well as the toe-bearing capacity (N_t) and combined shaft resistance factor (β) using the total stress method interpreted from Chesham (2005) were greater than those recommended by CFEM (2006).

6 CONCLUSIONS

PDA/CAPWAP testing and analysis provide an effective and cost-effective QA approach to ensure that the piles are installed to the design resistances. Using calculated geotechnical resistance at EOID, albeit conservative, proved to be an effective approach at installing piles to the design requirements as well as satisfying construction schedule and mobilization constraints.

The variability of the back calculated design parameters from the NEAHD and SWAHD projects, when compared to those recommended in CFEM (2006), reinforces the importance of pile load testing (static or dynamic) to supplement conventional geotechnical design methods.

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