Skin Friction and End Bearing Parameters of Cast-in-Place Concrete Piles Socketed into Calgary Bedrock

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ABSTRACT
The bedrock in the Calgary area belongs to the Paskapoo or Porcupine Hills Formations. Both formations are similar in depositional environment and geotechnical properties and consist of sandstone (SS), siltstone (SI), and claystone (CS). Due to the shallow depth of bedrock at several locations throughout the Calgary area, rock socket Cast-In-Place (CIP) concrete piles provide a preferred foundation option for major infrastructure projects in Calgary. This paper presents a comparison between the design parameters for rock socket CIP concrete piles calculated from the Unconfined Compression Strength (q_u) of the bedrock and the back calculated parameters from full scale instrumented pile load tests. The q_u of the bedrock employed in the comparison was obtained from laboratory tests on good quality bedrock core samples extracted from different areas in Calgary. The back calculated design parameters were obtained from two full scale load tests carried out on pre-production rock socket CIP concrete piles at the Stoney Transit Facility (STF) project in Calgary. The comparison showed that the skin friction and end bearing parameters of rock socket CIP concrete piles are underestimated when calculated from the laboratory q_u results of the bedrock core. The results suggest that the in-situ shear strength of Calgary bedrock is higher than the shear strength of the bedrock obtained from the laboratory tests by two to three times. This may be attributed to the potential disintegration and swelling of the bedrock taking place during coring, sampling, and transportation of the samples to the laboratory for testing.

ABSTRAIT
Le substrat rocheux dans les environs de Calgary est composé de la roche non-marine de la formation 'Paskapoo' ou 'Porcupine Hills'. Les deux formations sont généralement similaires dans leurs dépôts environnementaux et leurs propriétés géotechniques. La roche est composée de grès (SS), siltite (SI), and de l'argilite (CS). Puisque le substrat rocheux est peu profond à plusieurs endroits à Calgary, les pieux en béton coulés sur place encastres dans la roche sont l'option préférée pour les fondations des grands projets infrastructurels à Calgary. Cet article présente une comparaison entre les paramètres de conception des pieux en béton coulés sur place calculés en utilisant la force de compression non-confinée (q_u) de la roche et leurs valeurs calculées en arrière par l'analyse des résultats des essais de chargement à pleine échelle de pieux. Le 'q_u' du substrat rocheux utilisé dans la comparaison a été obtenu à partir de résultats de tests faite en laboratoire sur des échantillons de roche de bonne qualité extraits de différentes régions à Calgary. Les paramètres de conception calculés en arrière ont été obtenus à partir de deux essais de chargement à pleine échelle effectués sur des pieux de béton coulés sur place pré-production pendant le projet du Stoney Transit Facility (STF) à Calgary. La comparaison a montré que les paramètres de friction cutanée et de palier d'extrémité des pieux de béton coulés sur place enserrés dans le substrat rocheux sont fortement sous-estimés lorsqu'ils sont calculés à partir de valeurs de 'q_u' obtenu par un laboratoire. Les résultats suggèrent que la résistance au cisaillement in-situ de la roche à Calgary est de deux à trois plus élevée que celle du substrat rocheux obtenu à partir des résultats de 'q_u'. Cela peut être attribué à la désintégration et du gonflement potentielles du substrat rocheux au cours du carottage, de l'échantillonnage et du transport de l'échantillon au laboratoire pour analyse.

1 INTRODUCTION

The bedrock in the Calgary area is sedimentary and non-marine belonging to the Paskapoo or Porcupine Hills Formations (Prior et al. 2013). The distinction between the Paskapoo and Porcupine Hills Formations is primarily regional. Calgary is located at the transition zone between the two formations as shown in Figure 1. Both formations are similar in depositional environment and geotechnical properties; therefore, the bedrock formation in the Calgary area is referred to as the Paskapoo Formation in this paper. The Paskapoo Formation consists of flat lying to gently dipping SS, SI, and CS. All three bedrock units are non-marine, calcareous, interbedded and laterally/vertically discontinuous. The Paskapoo Formation has over 50 % of CS and SI deposits (Hamblin 2004). There is a weathered zone near the bedrock surface generally 1 m to 2 m thick. In some cases, the weathered zone is thicker than 2 m and may penetrate 4 m to 6 m below the bedrock surface.

The depth to bedrock across Calgary is highly variable. Bedrock was encountered near surface at some locations and at depths greater than 30 m below ground surface (BGS) in other locations. Shallow foundations on bedrock are generally feasible provided competent bedrock is within 3 mBGS. If the bedrock is deeper than 3 mBGS, rock socket piles are considered the feasible foundation alternative for infrastructure. Typically, the piles are socketed into bedrock between one to three times the pile diameter.

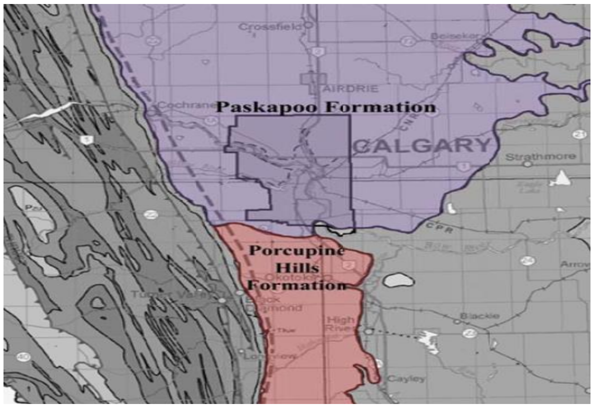


Figure 1: Calgary location with respect to the Paskapoo and Porcupine Hills Formations

Deep foundations in bedrock can be driven piles or CIP concrete piles. Driven piles are not suitable if the depth to bedrock is less than 6 mBGS. Additionally, piles driven into bedrock have many uncertainties with respect to type and quality of bedrock at the foundation level, penetration into the bedrock, and potential pile damage during driving. The capacity of the piles driven into bedrock and potential for pile damage during driving is typically assessed with Pile Dynamic Analyzer (PDA) testing on pre-production piles. Drilled CIP concrete piles are the preferred rock socket piles as the quality of the bedrock, drilling shaft diameters, and penetration depth into bedrock can be verified during pile installation.

Rock socket CIP concrete piles derive their capacity from shaft friction and end bearing in the bedrock. Other approaches suggest that the capacity of rock socket piles can be derived from shaft resistance only or end bearing resistance only. The later approach is considered a conservative approach.

The shaft resistance of the rock socket pile is estimated from the bond strength between the pile and the bedrock along the circumference of the rock socket. The magnitude of the shaft resistance is dependent on the quality and the roughness of the bedrock at the pile-rock interface (O'Neil and Reese 1999). The end bearing resistance of the pile is derived from the toe resistance of the pile; however, the applied load transmitted to the pile tips is typically 10 % to 20 % of the applied load and this ratio decreases as the slenderness ratio of the pile increases (Carter and Kulhawy 1988).

The shaft and end bearing resistances of rock socket CIP concrete piles are typically related to the q_u of the bedrock core (Nam et al. 2002). Therefore, the q_u of the rock mass should be evaluated carefully to obtain a representative value for estimating the skin friction and end bearing resistances.

Lo et al. (2009) carried out a laboratory research program to investigate the strength and deformation properties of CS in downtown Calgary. Lo and his co-authors described the sampling process of the CS as extremely difficult due to the fractured and highly fissile nature of the bedrock. They reported that all attempts to recover large samples by powered machinery were unsuccessful and resulted in bedrock disintegration. All successfully retrieved and tested samples were obtained by hand. The q_u of five square-section CS specimens extracted from the same site varied from 0.56 MPa to 1.23 MPa. The variability in q_u of the CS samples was attributed to the variability in sample size and slenderness ratio of the samples.

Crockford (2012) reviewed several field investigations performed in Calgary to compile the q_u data of the bedrock of the Paskapoo Formation and to estimate the percentages of each bedrock unit. The data analyzed in the study comprised of 145, 109, and 96 q_u tests on SS, SI, and CS, respectively. The median q_u of SS, SI and, CS were 23.7 MPa, 22 MPa, and 0.99 MPa, respectively and the observed percentages of SS, SI, and CS were 18.4 %, 33.8 %, and 47.8 %, respectively. Crockford concluded that the rockmass in Calgary is heterogeneous, inter-fingered, and discontinuous laterally; therefore, it is not possible to draw direct correlation between boreholes even within a specific project site.

Due to the heterogeneous rockmass, it is also not possible to predict if a pile at a particular location will be installed within one bedrock unit (CS, SI, or SS) or interbedded bedrock units (CS, SI, and SS). The uncertainty with respect to bedrock type at a particular pile location makes it challenging to estimate the shaft friction and end bearing of the piles. Accordingly, conservative parameters are provided for pile design; as a result, load carrying capacity of piles is underestimated.

This paper presents a comparison between the design parameters (shaft resistance and end bearing) of rock socket CIP concrete piles in the Paskapoo Formation estimated from the laboratory q_u of bedrock core and the back calculated design parameters from full scale pile load tests. This paper also presents the available correlations in literature used to estimate the shaft and end bearing resistances from laboratory q_u data.

2 BEDROCK STRENGTH IN THE CALGARY AREA

For this paper, the strength of the Calgary bedrock was evaluated from bedrock core q_u data. The bedrock core q_u data presented in this paper was obtained from six different project sites in north Calgary. The project sites are named A to F in this paper and the approximate project site locations are shown in Figure 2. The geotechnical investigations at these sites were completed by AECOM Canada Ltd. (AECOM) between 2013 and 2018. The projects included highway bridges and interchanges, a tunnel, several microtunnel, open cut installation, and a light rail transit.

The bedrock cores extracted from the six sites were obtained from depths varying from 5 mBGS to 95 mBGS. The bedrock core was tested to measure the q_u. Other parameters estimated from testing of the bedrock core are not presented in this paper. The embedment of piles is generally less than 30 m; therefore, q_u data for depths greater than 30 mBGS was not considered. In this study, the q_u of 154 intact non-weathered bedrock core samples collected from the six sites is presented. Out of the 154 bedrock core samples, 76 samples were CS (49 %), 29 samples were SI (19 %) and 49 samples were SS (32 %). Based on the results of the site investigations, the

Paskapoo Formation at these sites consisted of 44 to 56 % CS, 9 to 21 % SI, and 24 to 43 % SS.

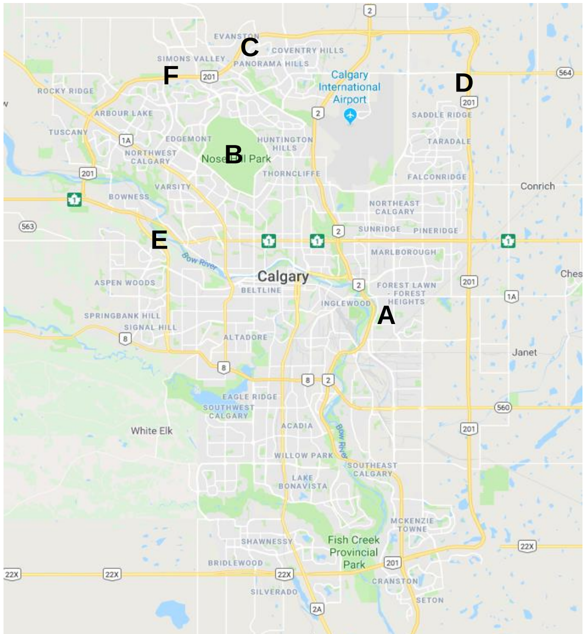


Figure 2: Project site locations (A-F) where bedrock core samples were obtained

The Canadian Foundation Engineering Manual (CFEM 2006) classifies bedrock (after Marinos and Hoek 2001) into seven grades based on q_u values. The grades range from Grade R0 – extremely weak bedrock with a q_u of 0.25 MPa to 1.0 MPa to Grade R6 – extremely strong bedrock with a q_u > 250. O'Neil and Reese (1999) classified the bedrock into cohesive soils ($q_u \leq 0.5$ MPa), intermediate geomaterial (0.5 MPa < $q_u \leq 5$ MPa), and sound rock (q_u > 5 MPa).

The q_u of the bedrock core presented in this study varied from 0.044 MPa to 69.89 MPa. The average and median q_u of the 154 core samples are 8.9 MPa and 4.4 MPa, respectively. The q_u of the 154 bedrock core samples collected from Sites A to F were classified in accordance with the CFEM (2006) and are presented in Figure 3.

Figure 3 shows that 53 % of the samples are extremely weak to very weak, 38 % are weak, and 9 % are medium strong to strong. Additionally, the bedrock from the 154 core samples was classified in accordance with O'Neil and Reese (1999) based on the q_u values of the core samples and is illustrated in Figure 4. Based on the O'Neil and Reese (1999) classification, 29 % of the samples are classified as cohesive material, 24 % are classified as intermediate geomaterial, and 47 % are classified as sound rock. It should be noted that, according to O'Neil and Reese (1999), 92 % of the 47 % cores classified as sound rock were from SS.

Based on the measured q_u, it is more practical to classify the bedrock in the Calgary area for design purposes in accordance with O'Neil and Reese (1999); i.e., intermediate geomaterial comprised of extremely weak to very weak CS interbedded with SS and SI with $q_u \leq$ 5MPa and sound bedrock with q_u > 5 MPa.

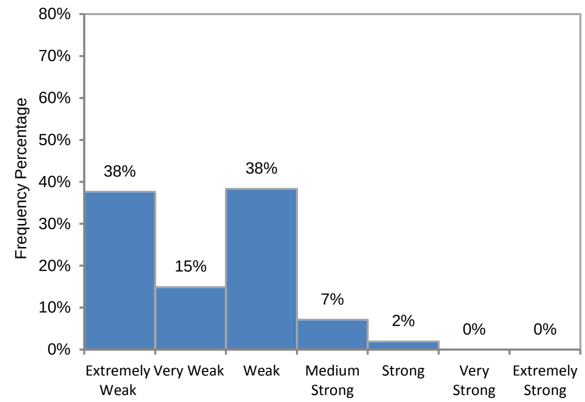


Figure 3: Distribution of measured q_u of bedrock core based on CFEM (2006)

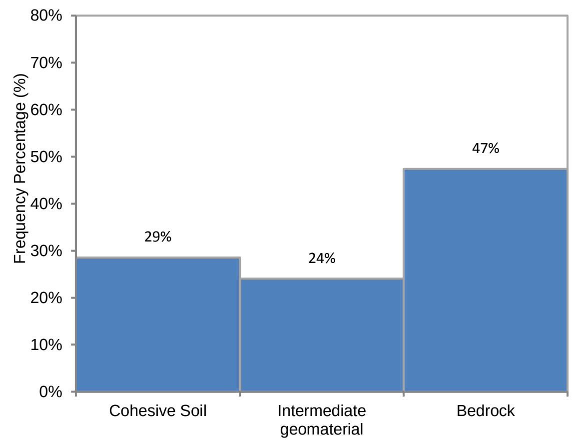


Figure 4: Distribution of measured q_u of bedrock core based on O'Neil and Reese (1999)

3 ESTIMATING SHAFT AND END BEARING RESISTANCES OF ROCK SOCKET CIP CONCRETE PILES

Several approaches have been developed in the past to estimate the skin friction and end bearing resistances for rock socket CIP concrete piles (CFEM 2006; O'Neil and Reese 1999; Carter and Kulhawy 1988; Rowe and Armitage 1984; and others). The majority of the approaches reported that the most important parameters that affect the capacity of piles in soft rock are the q_u, Young's Modulus, the roughness of the shaft walls, and the size, orientation and in-situ characteristics of the rockmass joints (Nam et al. 2002). Based on CFEM (2006), the ultimate shaft resistance of rock socket CIP concrete piles can be estimated using Equation 1:

$$\frac{f_{max}}{P_a} = b\left(\frac{q_u}{P_a}\right)^{0.5} \qquad [1]$$

Where: f_{max} is the ultimate average unit skin friction along the rock socket, P_a is the atmospheric pressure, and b is an empirical factor which can be taken as 0.63 as a conservative lower bound value. In cases where concrete compressive strength (f_c') is lower than the q_u of the bedrock the ultimate skin friction of the rock socket piles can be estimated using Equation 2:

$$f_{max} = 0.05\, f_c' \qquad [2]$$

The end bearing resistance of rock socket piles can be estimated according to CFEM (2006) using Equation 3:

$$q_{max} = 3 q_u K_{sp} d \qquad [3]$$

Where: K_{sp} is an empirical factor that varies from 0.1 to 0.4 depending on the bedrock discontinuity spacing; d is the depth factor = $1+0.4\, L_s/D_s \leq 3.0$, D_s is the diameter, and L_s is the length of the rock socket. CFEM (2006) stated that Equation 3 is generally not applicable for soft stratified bedrock such as shales, or limestones.

O'Neil and Reese (1999) suggested that for bedrock of q_u > 5MPa skin friction and end bearing resistances of the rock socket piles can be estimated from Equations 4 and 5, respectively:

$$\frac{f_{max}}{P_a} = 0.65\left(\frac{q_u}{P_a}\right)^{0.5} \qquad [4]$$

$$q_{max} = (1 \text{ to } 2)\, q_u \qquad [5]$$

O'Neil and Reese (1999) also suggested that the skin friction of the piles socketed into intermediate geomaterial can be estimated from Equations 6a and 6b:

$$f_{max} = f_{aa}; \qquad [6a]$$

$$f_{aa} = f_a \times (f_{aa}/f_a) \qquad [6b]$$

Where: f_a is the apparent ultimate unit side resistance and can be estimated from:

$$f_a = \alpha\, q_u \quad \text{for smooth socket} \qquad [7a]$$

$$f_a = q_u/2 \quad \text{for rough socket} \qquad [7b]$$

Where: α is a factor that varies from 0.08 to 0.5 depending on the interface angle of friction between the pile and the intermediate geomaterial, the normal effective pressure against the side of the rock socket, q_u of the intermediate geomaterial, and the mass modulus of elasticity of the intermediate geomaterial (should be less than 500 q_u); and (f_{aa}/f_a) is a factor that varies from 0.45 to 1.0 depending on the RQD of the intermediate geomaterial and the type of joints (open or closed) of the bedrock surrounding the socket. A threshold value of 0.5 is suggested for (f_{aa}/f_a) for extremely weak to very weak CS interbedded with SI and SS classified as intermediate geomaterial.

From above, the f_{max} for intermediate geomaterial can be estimated as:

$$f_{max} = (0.1 - 0.2)\, q_u \text{ for smooth socket} \qquad [8a]$$

$$f_{max} = 0.25\, q_u \quad \text{for rough socket} \qquad [8b]$$

For end bearing resistance of intermediate geomaterial, O'Neil and Reese (1999) suggested either to utilize the method given by Carter and Kulhawy (1988) or Equation 3 given by CFEM (2006). The method suggested by Carter and Kulhawy (1988) approximates the pile tip resistance as:

$$q_{max} = m_s\, q_u \qquad [9]$$

Where: m_s is a factor which is less than or equal 1.0 and depends on the quality of the rockmass, description and spacing of the rock joints, and the rock type. For fair quality CS interbedded with SS and SI m_s can be taken as 0.5 and for good quality rockmass m_s can be taken as 1.0.

Based on the q_u presented in the Section 2, the lower bound q_u for the CS interbedded with SI and SS is approximately 1MPa (classified as intermediate geomaterial). Using Equation 8 the estimated shaft resistance of rock socket piles in intermediate geomaterial varies from 100 kPa to 250 kPa depending on the roughness of the rock socket. Similarly, using Equation 9, the estimated end bearing resistance of the rock socket piles varies from 500 kPa to 1000 kPa. These resistances are considered lower bound values for the intermediate geomaterial. For bedrock with lower bound q_u of 5 MPa, the estimated shaft and end bearing resistances from Equations 4 and 5 are 400 kPa and 5000 kPa, respectively.

4 CASE STUDY FOR PILES INSTALLED IN CALGARY BEDROCK

4.1 Site Location and Project Background

Static load tests were carried out on two pre-production rock socket CIP concrete piles installed at the Stoney Transit Facility (STF) currently being constructed in northeast Calgary. The STF includes construction of a bus storage garage, a bus maintenance garage, an administration building, and a Compressed Natural Gas (CNG) facility in addition to other facilities. The locations of bus storage garage and bus maintenance garage are shown n Figure 5.

Based on subsurface conditions, rock socket CIP concrete piles were selected for this project. Two instrumented CIP piles (TP-01 and TP-02) were installed within the footprint of the bus storage garage (TP-01) and the bus maintenance garage (TP-02). The piles were tested to determine the geotechnical compressive load resistance of the rock socket piles. The locations of the two tested piles are shown in Figure 5. Keller Foundations Ltd. (Keller) was the piling contractor responsible for the installation of the two test piles and four reaction piles at each test pile location and the setup and performance of the pile load test. Tetra Tech Canada Inc. (Tetra Tech) was retained by

Keller to monitor the static load tests. AECOM, geotechnical consultants for the project, was invited by PCL to monitor the static pile load tests.

4.2 Subsurface Conditions

Thurber Engineering Ltd. (Thurber) performed a geotechnical investigation for the STF project in 2015 and submitted a geotechnical report to The City of Calgary. Forty nine (49) testholes were drilled to depths ranging from 1.5 mBGS to 11.9 mBGS. The soil stratigraphy was generally comprised of topsoil underlain by clay till underlain by bedrock. The bedrock was encountered at depths varying from 6.0 mBGS to 9.0 mBGS. The bedrock consisted of CS with interbedded layers of SI and SS. The subsurface conditions at the test pile TP-01 and TP-02 locations were inferred from nearby testholes TH15-23 and TH15-16, respectively. The locations of the testholes with respect to the test piles are shown on Figure 5.

The subsurface stratigraphy observed during pile installation at TP-01 consisted of a 7.9 m thick layer of clay till underlain by bedrock. The bedrock consisted of extremely weak CS with a 300 mm thick layer of more competent SS at depth of 9.5 mBGS. The boreholes and probeholes drilled by Keller within 6 m of TP-01 in the presence of Tetra Tech indicated that the bedrock is highly variable and the depth to the top of the bedrock varied from 7.2 mBGS to 9.5 mBGS (Tetra Tech 2017).

The subsurface stratigraphy observed during pile installation at TP-02 consisted of a 6.8 m thick layer of clay till underlain by bedrock. The upper 1.8 m of bedrock was relatively competent SS underlain by extremely weak CS. Based on the Keller boreholes and probeholes drilled within 6 m of TP-02, the depth to the top of the bedrock varied from 6.1 mBGS to 9.4 mBGS. Furthermore, the presence/absence of competent bedrock layers varied significantly between the Keller borehole/probehole locations (Tetra Tech 2017).

4.3 Pile Load Test Installation, Instrumentation, and Setup

Test piles TP-01 and TP-02 were installed on February 15, 2017 to depths of 10.3 mBGS and 12 mBGS, respectively. Both test piles had a diameter of 610 mm. Four reaction piles were installed for each pile load test as shown on Figure 6. All reaction piles were 762 mm in diameter and installed to a depth of 14.5 mBGS. The test piles extended approximately 300 mm above ground level. The upper portion of the test piles were cased with a 900 mm long, 610 mm (outside diameter) steel casing to minimize the risk of failing the piles structurally either above ground or just below ground surface. Figure 7 presents a schematic of the test pile TP-01 and TP-02 instrumnetation. A concrete mix with 7-day and 56-day strengths of 58 and 70 MPa, respectively, was used for both test piles.

Five pairs of strain gauges (total of 10 strain gauges) were installed on the reinforcing steel cages for each test pile, as shown on Figure 7. The strain gauges (or sister bars) comprised vibrating wire rebar strain gauges (Model 4911-4) supplied by Geokon Incorporated. The strain gauges were installed in the test piles to measure the strain developed along the pile lengths during the load tests. Pile head movements were measured using four linear potentiometers (Model LDS-1000) supplied by RST Instruments Ltd. The potentiometers were mounted on a steel bearing plate placed directly on top of the test pile. As a check, pile head movement was also measured using two dial gauges mounted on the steel bearing plate.

Load was applied by a hydraulic jack and was measured using a strain gauge load cell transducer (Model SGS-2250), both supplied by Keller. The load tests for TP-01 and TP-02 were conducted on February 24 and 28, 2017, respectively. The compressive load was applied by an 8,900 kN (1,000 ton) hydraulic jack reacting against the main reaction beam as shown on Figure 8. A datalogger was used to record the measurements of the strain gauges and linear potentiometers. A laptop computer was connected to the datalogger for data acquisition and for real-time monitoring of the instruments. The readouts from the strain gauges and potentiometers were recorded every 10 seconds and were later reduced and analyzed following the load tests.

4.4 Pile Load Test Results

The ultimate pile capacity for TP-01 was 4,500 kN based on experience from previous load tests. The test load was applied in 225 kN increments with a uniform hold time of eight minutes per increment, with a 60 minute hold time at a load of 4500 kN. After the one hour hold period, pile loading continued to approximately 6750 kN where the rated capacity of the reaction system was reached. The pile head deflection at the maximum applied load was about 15.7 mm.

For TP-02, the compressive load at the pile head was applied in 250 kN increments to the estimated ultimate capacity of 5,000 kN and was held for one hour. Pile loading continued to the rated capacity of the reaction system of about 6,750 kN. The pile head deflection at the maximum applied load was about 8.1 mm. Both piles were unloaded in four equal stages after reaching the rated capacity of the reaction system. The load displacement curves for test piles TP-01 and 02 are shown on Figure 9.

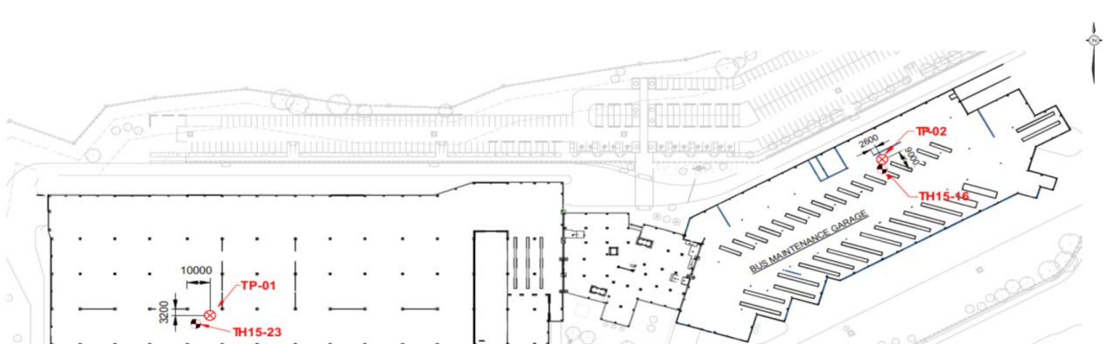
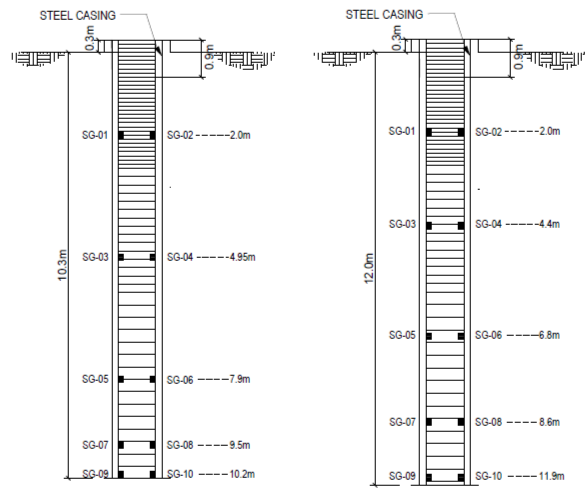


Figure 5. Locations of the STF buildings and the two test piles – TP01 and TP02

Figure 6: Pile load setup with four reaction piles

Figures 10a and 10b present the interpreted mobilized shaft resistances versus pile head displacements for TP-01 and TP-02, respectively. Both plots show that the mobilized shaft frictions in clay till are plateaued while the mobilized shaft frictions in the bedrock continue to increase with pile head deflection. Furthermore, relatively low end bearing resistances, in the order of 1,200 to 1,400 kN in bedrock, were interpreted for both test piles. The relatively low end bearings and the still rising shaft frictions in the bedrock indicate that geotechnical resistances of both piles were not fully mobilized in the rock socket. For TP-02, high shaft friction was interpreted to be in the “hard layer” of bedrock at depths ranging from 6.8 mBGS to 8.6 mBGS, which corresponds well with the drilling response during test pile installation.

The relatively low shaft friction in the bedrock below the “hard layer” is likely because the shaft friction in this unit not being fully mobilized under small pile head deflection.



TP-01 TP-02

Figure 7: Schematic sections of TP-01 and TP02

Comparison between existing boreholes, the probeholes, and drilling responses from test pile installations indicate that the subsurface conditions and the bedrock depths and strength are highly variable across the site, even over a short distance. The soil/bedrock conditions encountered at the test pile locations do not necessarily represent the credible lower bound conditions across the site. In addition, the hard layer(s) in the bedrock encountered at test pile TP-02 may not exist at other locations, and even if these layers do exist, the thickness and depth of the hard layer(s) cannot be delineated. As a result, considerable engineering judgement is needed to derive the recommended parameters for production pile design (Tetra Tec 2017). Based on the pile load test results, Tetra Tech (2017) recommended shaft friction and end bearing resistance parameters provided in Table 1 for design of the production piles.

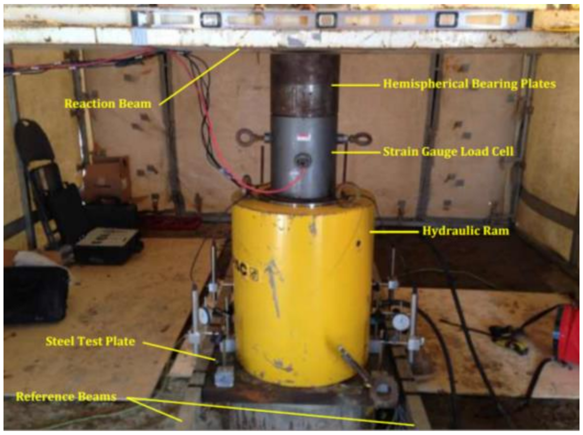


Figure 8: Compressive pile load test setup

Table 1: Recommended Shaft and End Bearing Resistances for Rock Socket Piles (Tetra Tech 2017)

Soil Type	Ultimate Shaft Friction (kPa)	Ultimate End Baring (kPa)
Desiccation Zone (0 - 2 m)	0	0
Clay Till	80	1200
Weathered Bedrock	200	2500
Competent Bedrock	350	5000

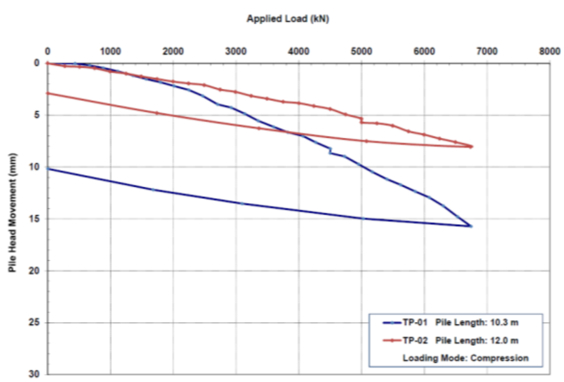


Figure 9: Pile head movement versus applied load

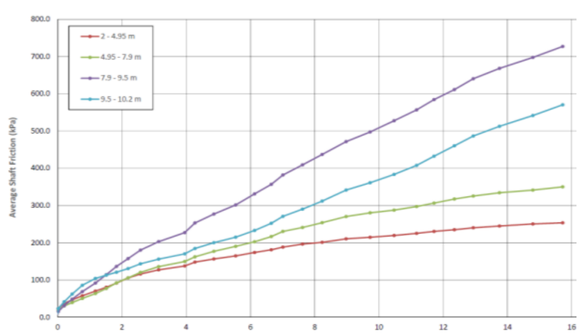


Figure 10a: Pile head displacement versus average shaft friction TP-01

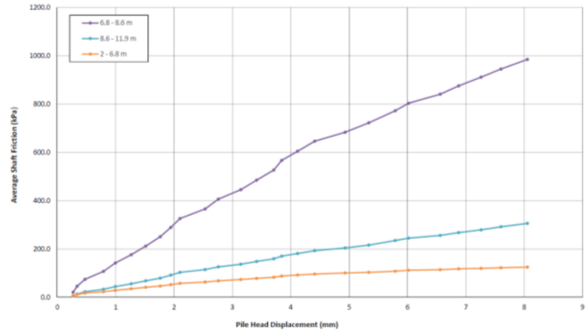


Figure 10b: Pile head displacement versus average shaft friction TP-02

5 DISCUSSION OF THE PILE LOAD TEST RESULTS

Based on the average skin friction shown in Figures 10a and 10b for the test piles, the shaft resistance of the competent bedrock varied from 350 kPa to over 1000 kPa. Furthermore, the load transfer to the bedrock at the pile tips from the maximum applied load of 6700 kN is between 1200 kN and 1400 kN; for 610 mm piles, this load corresponds to end bearing resistance between 4100 kPa and 4800 kPa. It should be noted that neither of the two piles reached geotechnical failure at this loading level. Having said that, an ultimate shaft resistance of 350 kPa and end bearing of 5000 kPa provided in Table 1 are considered the lower bound resistances for the tested piles. The lower bound resistances are provided for pile design due to the variability in the soil/bedrock conditions at the site, which is common in Calgary.

Although the pile design parameters in Table 1 provide the lower bound values, these values are higher than the estimated values for intermediate geomaterial comprised of CS interbedded with SI and SS given in Section 2. The shaft resistance of the competent bedrock provided in Table 1 is 1.4 to 3.5 times higher than the shaft resistance estimated and end bearing is approximately 5 times higher than the estimated resistances for intermediate geomaterial. On the other hand, if CS interbedded with SI and SS is classified as bedrock (according to O'Neil and Reese 1999) with a lower bound q_u of 5 MPa, the shaft and end bearing resistances in Table 1 are equal to or slightly less than the lower bound values estimated for bedrock.

The difference between the estimated and back calculated pile design parameters of the CS interbedded with SI and SS can be attributed to the fact that estimated pile design parameters are based on the q_u and quality of the bedrock core. The bedrock in the Calgary area is sedimentary and mainly consists of CS interbedded with SS and SI which disintegrates when exposed to air and swells when the overburden pressure is removed. This disintegration and swelling lead to low strength and low quality core, even when all precautions were taken during coring, sampling, and transporting of the bedrock core samples. The strength of the rockmass in Calgary is typically two to three times higher than the core strength. Only SS not interbedded with CS and SI may have a rockmass strength that is comparable to its core strength; however, it has been proved from several sites across Calgary that bedrock is laterally and vertically discontinuous and the likelihood of continuous SS, SI, and CS is very low.

6 SUMMARY AND CONCLUSIONS

The bedrock in Calgary is unique and poses a challenge to designers as it is comprised of sedimentary interbedded layers of CS, SI, and SS which are discontinuous laterally and vertically. The q_u of the bedrock in Calgary was evaluated in this study from 154 core samples obtained from six different sites across Calgary from depths varying from 5 mBGS to 30 mBGS. According to the O'Neil and Reese (1999) classification, more than 50 % of the bedrock samples are classified as intermediate geomaterial with q_u less than 5 MPa. Therefore, it is more practical to classify the bedrock in the Calgary area into two types: intermediate geomaterial comprising extremely weak to very weak CS interbedded with SI and SS (q_u < 5 MPa) and sound bedrock with $q_u \geq 5$ MPa. The pile design parameters for each type can be estimated following the approaches suggested by O'Neil and Reese (1999).
The results of full scale pile load tests on two 610 mm rock socket CIP concrete piles were presented and analyzed. The back calculated lower bound skin friction and end bearing resistances of the two test piles were 1.4 to 5 times higher than the estimated resistances utilizing lower bound q_u for intermediate geomaterial.
The lower estimated values are due to disintegration and swelling of the bedrock core during sampling, transportation, and testing in the laboratory which lead to low strengths and accordingly lower estimated pile design parameters. In other words, the strength of the rockmass in the Calgary area is higher than the strength of the bedrock core by two to three times. It is recommended to use an average rather than lower bound values of q_u for bedrock core to estimate representative skin and end bearing resistances for rock socket CIP concrete piles in Calgary and optimize the pile design by pile load testing whenever feasible.

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