A modern approach to the design of driven precast-prestressed concrete piles in Winnipeg

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ABSTRACT
In the 1960's, allowable loads were tabulated in a Winnipeg by-law for driven precast-prestressed concrete hexagonal piles. These historical values were used successfully for decades and it is common in current local practice to consider these tabulated loads as the basis for limit states design. Therefore, there is a need to reconsider the methodology for the design of this pile type in Winnipeg by separately analyzing pile capacity and settlement. Static load test results from 1969 were analyzed with a modern, limit states design approach. A load-transfer model was calibrated and pile capacity and settlement were then calculated for the test site. The current practice based on tabulated working loads was found to be conservative. It is recommended that site specific analyses be performed to design this pile type and that the historical tabulated working loads are considered carefully for application in the limit states design approach.

RÉSUMÉ
Dans les années 1960, les charges admissibles ont été compilées dans un règlement municipal de Winnipeg pour les pieux battus hexagonaux en béton armé préfabriqué et précontraint. Ces valeurs historiques ont été utilisées avec succès pendant des décennies et il est courant, dans la pratique locale actuelle, de considérer les données compilées comme la base de la conception aux états limites. Par conséquent, il est nécessaire de réactualiser la méthodologie de conception de ce type de pieu à Winnipeg par une analyse séparée de la capacité portante et du tassement des pieux. Les résultats des essais de chargement statique depuis 1969 ont été analysés selon une approche moderne de conception aux états limites pour calibrer un modèle de transfert de charge. La capacité portante ainsi que le tassement d'un pieu ont été ensuite calculés pour le site d'essai. La pratique actuelle basée sur les charges de service compilées s'est avérée conservatrice. Il est recommandé d'effectuer des analyses spécifiques au site pour concevoir ce type de pieu et de prendre en compte les charges de service historiques compilées pour une application dans l'approche de conception aux états limites.

1 INTRODUCTION

Driven precast-prestressed hexagonal concrete (PPHC) piles have been extensively used in Winnipeg, Manitoba for decades (Baracos et al. 1983). The former working stress design methodology led to successful performance of this pile type and local engineers developed judgement from this experience. Building codes in Canada including the National Building Code of Canada (National Research Council of Canada 2015) and the Canadian Highway Bridge Design Code (Canadian Standards Association 2014) have since adopted the requirement for Limit States Design for deep foundations. Limit States Design is intended to provide an adequate level of safety with the geotechnical ultimate limit state (ULS) and adequate performance with the serviceability limit state (SLS) (Canadian Geotechnical Society 2006). This requires a distinct change in design process to address both pile capacity and settlement. Local practice has adapted however by aiming to achieve a consistent design to the previous working stress design method rather than aiming to address the limit states criteria. Therefore, there is a need to review and potentially improve the current Limit States Design practice in Winnipeg for driven PPHC piles.

PPHC piles have been manufactured in Winnipeg since 1961 (M. Block and Associates Ltd. et al. 1978). Shortly after in 1964, the Winnipeg By-Law 711 was introduced which differed from the National Building Code of Canada at the time by tabulating allowable loads for driven PPHC piles (Skaftfeld 2014). These loads were 450 kN, 635 kN, and 800 kN for 305 mm (12”), 356 mm (14”), and 406 mm (16”) diameter piles, respectively. The factor of safety is often assumed to be 2.5 for the stated allowable loads (Skaftfeld 2014). These allowable loads will be referred to as the “historical allowable loads”. The prescribed values were in effect until the 1977 Manitoba Building Code was introduced and all references to the tabulated values were deleted (M. Block and Associates Ltd. et al. 1978).

The Manitoba Precast Prestressed Concrete Pile Capacity Study (M. Block and Associates et al. 1978) summarizes the results of three independent studies on the capacity of PPHC piles. This includes pile load tests carried out by M. Block and Associates Ltd., wave equation analysis performed by R.M. Hardy and Associates, and a pressuremeter investigation carried out by Klohn Leonoff Consultants Ltd. This report concluded that it is reasonable to state a capacity that is 30% greater than the historical capacities and capacities could be 80 – 90% greater. The exact increase however will depend on several factors including site subsurface conditions, pile length and size, driving energy, cushion block assembly, and the driving refusal criteria (M. Block and Associates Ltd. et al. 1978). Local foundation design practice largely continued to use the lower historical capacities however based on successful past performance (Skaftfeld 2014).

Local practice in Winnipeg has adapted to Limit States Design based on the historical allowable loads. The SLS load (Q_{SLS}) is often stated to be equivalent to the historical allowable loads along with a maximum expected settlement. This method does not address the true intent of

SLS unless a settlement analysis is performed and considers the settlement criteria of the structure. The ULS capacity (Q_{ULS}) is often stated as 2.5 times the historical allowable load. A consistent city-wide Q_{ULS} does not truly consider the capacity of a pile since it does not consider the site specific subsurface conditions, pile length and size, or driving methodology.

There is an opportunity to improve the Limit States Design practice in Winnipeg by better addressing the true intent of ULS and SLS design. The purpose of this paper is to illustrate how driven PPHC piles can be designed in consideration of the pile capacity and settlement. This paper summarizes an analysis of pile load test data from the Manitoba Precast Prestressed Concrete Pile Capacity Study. A load-transfer analysis was performed to estimate shaft load-movement (*t-z*) and toe load-movement (*Q-z*) functions that are representative of the load test results. A load-transfer analysis was then performed to calculate Q_{SLS} and Q_{ULS} for the test pile site. Concepts from the Unified Design method (Fellenius 1984, 2018) and recommendations from the Minnesota Department of Transportation (MnDOT) Geotechnical Manual (MnDOT 2017) are used in performing the load-transfer analysis.

2 HISTORICAL LOAD TESTING PROGRAM

The pile load testing described in the Manitoba Precast Prestressed Concrete Pile Capacity Study was performed in 1969 by Conforce Ltd. (now AP Infrastructure Solutions LP) on their property in Winnipeg. Static load tests were completed on six piles with two being 305 mm (12") diameter, two being 406 mm (16"), and two being 559 mm (22") diameter.

The soil at the test site consisted of approximately 15.9 m of soft to firm clay overlying a 'putty till' changing to a dense till with depth. Standard Penetration Test *N*-values were reported in the range of 100 to 250 in the dense till. The groundwater level was noted to be approximately 1.2 m below ground surface.

The piles were pre-bored to 7.6 m (25') and were driven to approximately 18.3 m (60') below ground surface. These piles were driven with a Linkbelt 520 pile driving hammer rated at a maximum energy of 40.7 kJ per blow. The final blow counts per 25 mm varied and are summarized in Table 1. Baracos et al. (1983) reported typical refusal blow counts used in local practice which ranged from 3 to 5 blows per 25 mm for 305 mm piles, 7 to 10 blows per 25 mm for 356 mm piles, and 9 to 15 blows per 25 mm for 406 mm piles. Higher values in these ranges are typical for the Linkbelt 520. A typical final penetration set currently used for preliminary design in Winnipeg is 5, 8 and 12 blows per 25 mm for the 305, 356, and 406 mm diameter sizes, respectively.

The load testing procedure was described in the Manitoba Precast Prestressed Concrete Pile Capacity Study. The test piles were loaded in 89 kN (10 ton) increments for initial stages. The increments were then

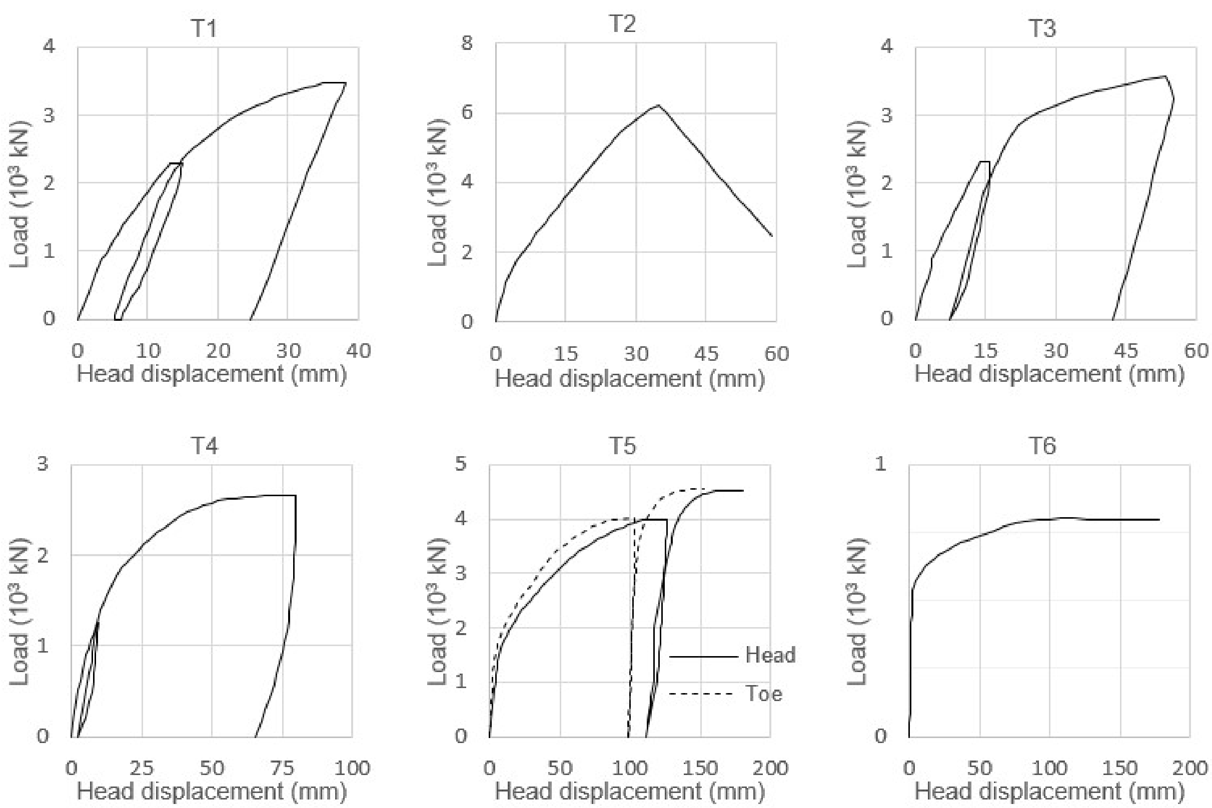


Figure 1: Static load test results from the Manitoba Precast Prestressed Concrete Pile Capacity Study (data from M. Block and Associates Ltd. et al. 1978)

increased to 445 kN (50 ton) or 267 kN (30 ton) for the 305 mm piles. Each load increment was held for two hours. When the load reached what was believed to be twice the design load, it was held for 24 hours. The load was removed in four equal increments with half hour time intervals. The pile was then reloaded in four equal increments before resuming the previous load increments. The static load test results shown in the report were digitized and are shown in Figure 1. A load holding period was noted prior to final unloading of Pile T1 and at the end of testing Pile T6. Compression failure of the concrete was noted for Pile T2 where the load decreases. A tip measuring rod was placed in the hollow shaft of Pile T5 and head load versus toe movement is also shown.

Table 1. Final penetration set from 1969 test pile program (from M. Block and Associates Ltd. et al., 1978)

Pile #	Diameter (mm)	Final blows / 25 mm
T1	406	18
T2	559	21
T3	406	11
T4	305	6
T5	559	8
T6	305	3

3 LOAD-TRANSFER PILE ANALYSIS

The static load test results were used to calibrate *t-z* and *Q-z* functions for the test site. A load-transfer analysis was then performed to calculate Q_{SLS} and Q_{ULS} values for a 406 mm diameter pile at this site.

3.1 Model Calibration

The compression of the piles was not measured directly during the static load tests. The toe movement of Pile T5 was measured, however Fellenius (2018) has stated that pile shortening must be measured directly to estimate the average load in the pile. This is because small movements of the reference beam will result in large errors in calculating pile compression. Therefore, the proportion of shaft versus toe resistance can't be directly interpreted from the load test results themselves. To perform the load-transfer analysis, reasonable *t-z* functions were estimated and *Q-z* functions were then calibrated such that the modeled load-movement of the pile head reasonably matched the static load test results. RSPile (Rocscience Inc., 2018) was used to numerically calculate the axial pile force and settlement distribution with depth for various applied loads.

The hexagonal pre-cast concrete piles were modelled as an elastic material with Young's Modulus of 34,473,786 kPa (5,000,000 psi) as reported from concrete

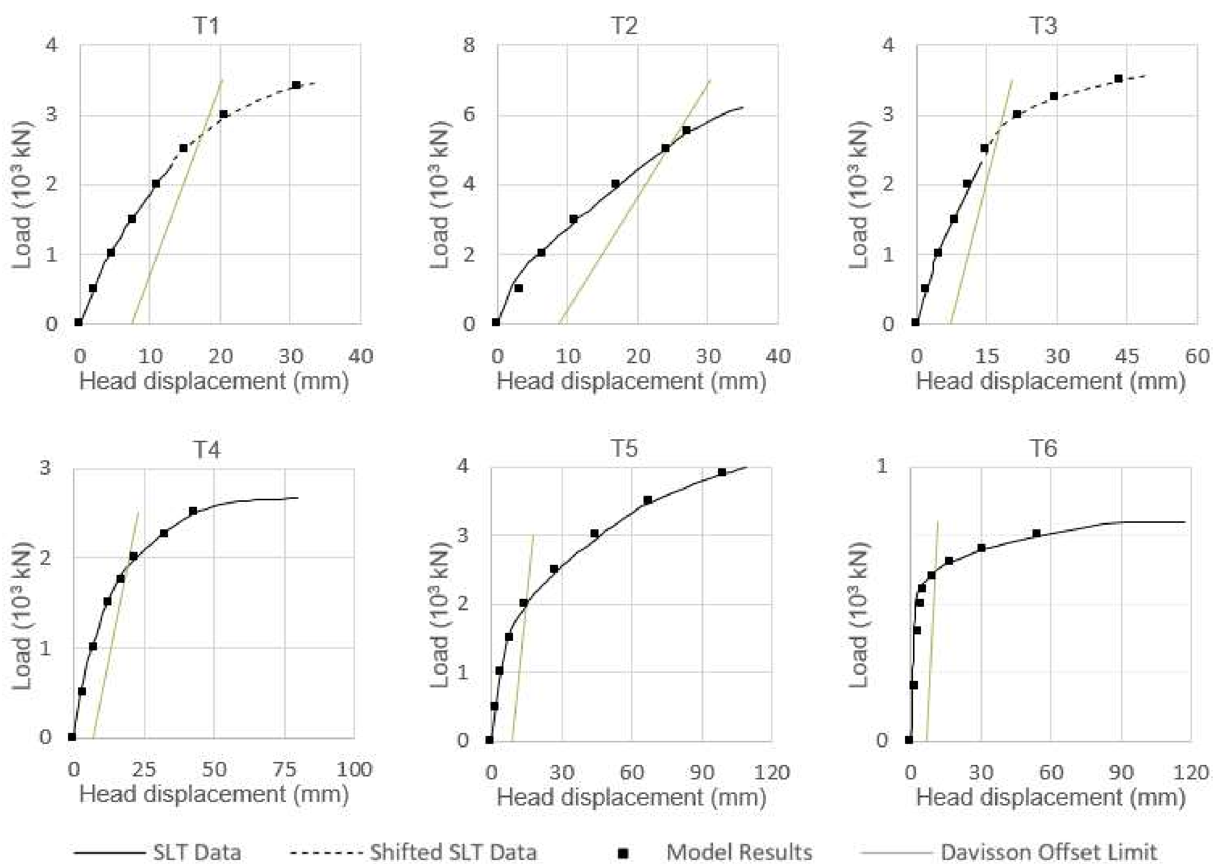


Figure 2: Load-transfer model results versus static load test (SLT) results (data from M. Block and Associates Ltd. et al. 1978)

testing of the test piles (M. Block and Associates Ltd. et al. 1978). The pile geometry input into the model for the various piles types is shown in Table 2. The 559 mm diameter piles were cast with a cylindrical hollow core so the cross sectional area along the shaft differs from the end bearing area at the toe.

Table 2. Geometric properties of modelled piles

Pile #	Diameter (mm)	Perimeter (m)	Area (m^2)
T4, T6	305	1.056	0.0806
T1, T3	406	1.408	0.1432
T2, T5	559	1.935	0.1710[1] / 0.2703[2]

[1]cross sectional area of hollow pile shaft

[2]end bearing area at pile toe

The *t-z* functions for the pile shaft were assumed to behave elastic-perfectly plastic. Shaft resistance has been observed to fully mobilize with several mm of relative pile versus soil displacement (Fellenius 2006). Therefore, the maximum shear resistance was modelled to occur at 2.5 mm of displacement. The beta-method was used to calculate the maximum shear resistance. The upper 7.6 m was pre-bored prior to pile installation and the shaft resistance was considered to be negligible along this portion. Beta was assumed to be 0.25 in the clay and is within the range stated in the Canadian Foundation Engineering Manual (Canadian Geotechnical Society 2006). Beta was assumed to be 0.46 for the till consistent with the American Petroleum Institute (API) recommendations for very dense silt-sand (API 2007). The unit weight of the clay was input as 17 kN/m^3 and the till was input as 22 kN/m^3. Custom *Q-z* functions were input and adjusted such that the modelled load-movement behaviour of the piles approximated the measured behaviour during the static load tests.

The modelled pile head load-movement is shown in Figure 2 and is compared to the actual static load test data. The static load test results for piles T1 and T3 have been adjusted by shifting the displacement data after a load holding period and unloading/reloading cycle. The modelled behaviour is observed to reasonably match the actual data.

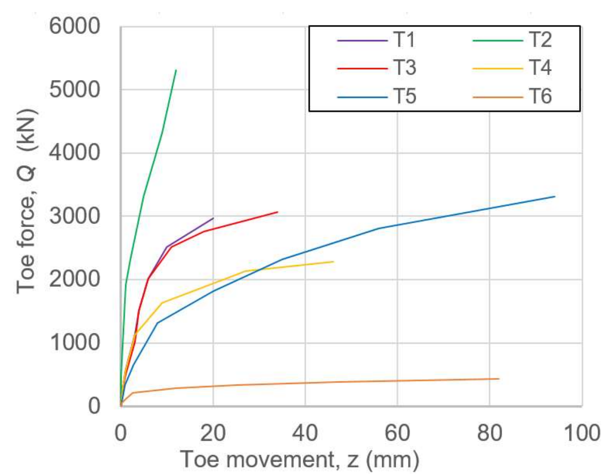
Each of the custom *Q-z* functions is plotted on Figure 3a. Figure 3b shows the functions after normalizing the toe load to the toe area and normalizing the toe displacement to the pile diameter. Piles T1, T2, T3, and T4 are observed to have relatively consistent toe stiffness when normalized. Piles T5 and T6 are observed to have relatively softer toe stiffness. The *Q-z* curve developed from the API (American Petroleum Institute 2007) method for sand is shown for reference with toe bearing capacity factor (N_t) values of 160 and 45 for comparison.

3.2 Calculation of SLS load

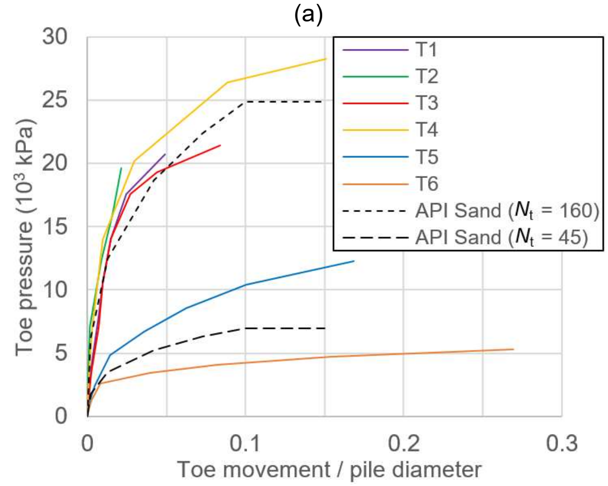
The calibrated model was used to calculate Q_{SLS} of a 406 mm diameter driven PPHC pile at the test site. This analysis is based on a hypothetical structural requirement of a maximum 15 mm of allowable settlement under expected service loads. Variables that will influence the long term settlement include the service load, the shear strength and mobilization of resistance along the shaft, the stiffness of the toe response, Young's modulus of the pile, and the distribution of relative shaft versus soil displacement. The variability of these factors was considered in calculating an expected range for Q_{SLS} to satisfy the maximum settlement of 15 mm.

The pile toe stiffness was observed to have a large range from the model calibration. The *Q-z* function was modelled using the API method for sand with N_t of 160 to represent the expected behaviour and N_t of 45 to represent a worst case scenario. The analysis was also performed with N_t of 102.5 as the midway point between expected and worst-case scenarios.

It is challenging to predict the long-term distribution of negative skin friction and positive shaft resistance. Negative skin friction has been documented to occur on all piles however (Fellenius 1998). For driven piles in particular, the soil/rock below the toe pushes upward after



(a)



(b)

Figure 3: Calibrated toe stiffness; (a) *Q-z* function, (b) normalized *Q-z* function

driving and the pile elastically decompresses which is resisted by negative skin friction (Briaud and Tucker 1984). Therefore the distribution of negative skin friction was considered in this analysis. A method from the MnDOT Geotechnical Manual (MnDOT 2017) was used to estimate the distribution of negative skin friction and positive shaft resistance for expected conditions where ground settlement is small. This method assumes the proportion of mobilized shear stress along the shaft. The shaft shear stress is assumed to be zero at the pile head and linearly increasing to 25% of maximum shaft shear in the negative direction at the base of the compressible soil layer. The shear stress then linearly increasing to 100% positive shaft resistance at the pile toe. A conservative scenario was also analyzed where significant negative skin friction develops along the pile shaft. This analysis considered zero ground settlement in the till and settlement in the clay linearly increasing from 0 mm at the clay-till interface to 300 mm at ground surface. This is considered an unlikely and worst-case scenario.

A sensitivity analysis was not performed on the *t-z* functions because they were assumed in order to develop the expected range of *Q-z* functions. It was assumed that for long term conditions that the soil along the pre-bored portion of the pile shaft would come in contact with the pile. Beta was modelled as 0.25 consistent with the clay below.

A sensitivity analysis was also not performed on the Young's Modulus of the pile because the pre-cast piles are expected to be manufactured in controlled conditions.

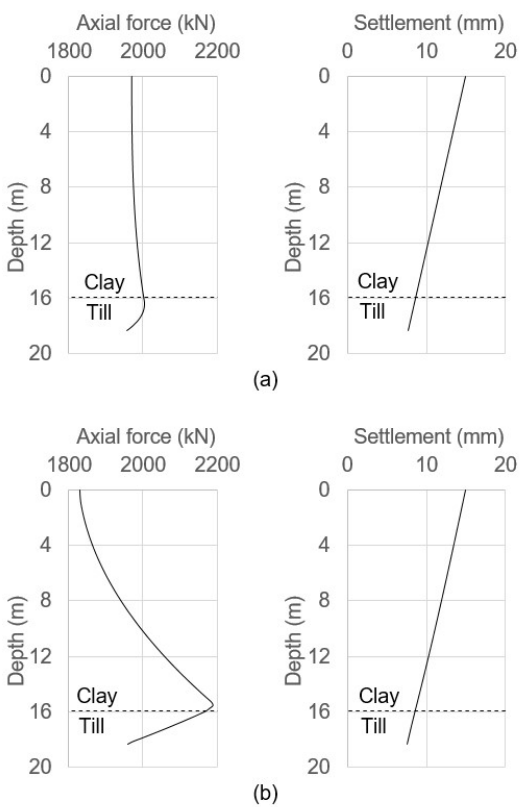


Figure 4: Force and settlement distribution for 15 mm of head settlement; (a) with expected negative skin friction, (b) with worst-case negative skin friction

The settlement of the pile head was calculated from the sum of the toe penetration and elastic shortening of the pile to determine a load that results in approximately 15 mm of settlement. Continuity of the toe force and toe movement based on the *Q-z* function was maintained, consistent with the settlement analysis of the Unified Design method (Fellenius 2018). The distribution of force and settlement of the piles for the expected and worst case distribution of negative skin friction is shown in Figure 4. This shows the distribution with the expected toe stiffness where N_t is 160. For the expected distribution of negative skin friction, a sustained load of 1,970 kN results in a settlement of 15 mm. For the worst case distribution of negative skin friction, a sustained load of 1,830 kN results in a settlement of 15 mm.

The sensitivity of Q_{SLS} for the varying *Q-z* functions is shown in Figure 5. The toe stiffness is observed to have significantly more impact on Q_{SLS} than the distribution of negative skin friction for this site. The typical Q_{SLS} value of 800 kN used in local practice for a 406 mm diameter pile is shown in Figure 5 for reference.

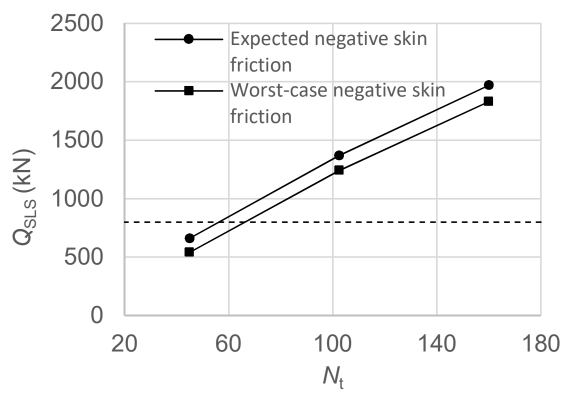


Figure 5: Sensitivity of Q_{SLS} for 15 mm of head settlement with varying toe stiffness

3.3 Calculation of ULS capacity

There are numerous definitions and methods to determine the capacity of a pile. Pile capacities were calculated for the 406 mm piles, Piles T1 and T3, from the load test results and the calibrated model. The Davisson Offset (Davisson 1973) method was used due to its prevalence in modern practice from dynamic testing. A preferred definition by Fellenius (2014) where capacity is defined as 30 mm of toe displacement was also calculated. Calculating capacity with this definition required the use of the calibrated RSPile model. Values of capacity reported in the Manitoba Precast Prestressed Concrete Pile Capacity Study were determined by the Illinois Highway Department method where a graphical analysis is performed to find the point where the first progressive yielding occurs (Housel 1966). The capacities as determined from these methods are summarized in Table 3.

Table 3. Capacity of Piles T1 and T3 for various methods

Pile #	Illinois Highway Department[1]	Davisson Offset	30 mm toe movement
T1	2,580 kN	2,720 kN	>3,460 kN[2]
T3	2,616 kN	2,790 kN	3,480 kN

[1]from Manitoba Precast Prestressed Concrete Pile Capacity Study
[2]maximum load during load test. Calculated toe movement was less than 30 mm.

The Illinois Highway Department method capacities were determined from the second loading cycle. The Davisson Offset method capacities were determined from the adjusted static load test results in Figure 2. The force and settlement distribution of Pile T3 where toe movement is equal to 30 mm is shown in Figure 6. The shaft resistance was considered to be negligible in the pre-bored portion of the pile. A modelled capacity for 30 mm of toe movement was not calculated for Pile T1 because it would require extrapolating from the maximum load during load testing. The maximum load during the test was 3,460 kN and the calculated toe displacement is less than 30 mm at that load increment.

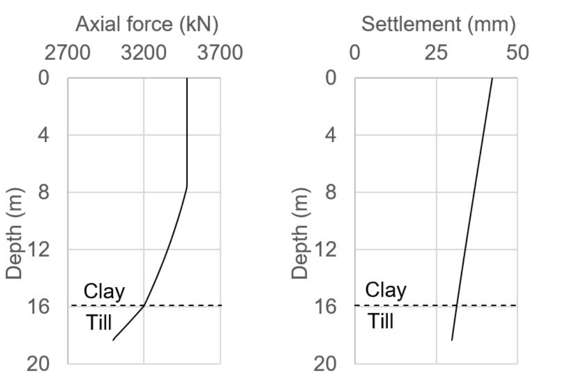


Figure 6: Force and settlement distribution of Pile T3 for 30 mm of toe movement

4 DISCUSSION

The typical Q_{SLS} value of 800 kN for a 406 mm diameter PPHC pile is much lower than the calculated expected Q_{SLS} of 1,970 kN for 15 mm of long term settlement at the test site. The analysis performed for serviceability conditions indicated that the range of toe stiffness from expected to worst-case scenario has a significant impact on the expected settlement. Piles T5 and T6 were driven with lower final blow counts compared to the other piles and compared to typical local practice. These two piles had softer toe stiffness. Q_{SLS} was less than 800 kN when calculated with the worst case toe stiffness. These results indicate a correlation between pile driving methodology and pile performance. The distribution of negative skin friction was also found to affect the calculated pile settlement. Q_{SLS} was less sensitive to the variability of negative skin friction distribution compared to the variability of toe stiffness. Accurately predicting the long term settlement to mm accuracy is not a realistic expectation. Performing a settlement analysis by considering the realistic long term load-transfer including sensitivity analyses on key variables provides insight into the expected range of performance. Settlement is not likely to govern the design when the pile toe has a stiff response. This is consistent with the MnDOT Geotechnical Manual statement that serviceability is usually a secondary consideration for piles driven to hard bearing layers (MnDOT 2017).

Driven PPHC piles in Winnipeg are often designed with a Q_{ULS} of 2.5 times the historical allowable load of 800 kN which is equal to 2,000 kN. The capacities at the test site were measured and calculated to be greater. The capacity as determined from the Illinois Highway Department method is approximately 2,600 kN. This corresponds to the 30% increase over historical practice recommended by Manitoba Precast Prestressed Concrete Pile Capacity Study. The capacity as determined from the Davisson Offset method was slightly higher at approximately 2,750 kN. The Davisson Offset provides a failure value that is conservative and does not consider a plunging failure of the pile (Canadian Geotechnical Society 2006). It should be noted that this method is not suitable for testing procedures that include unloading and reloading cycles however (Canadian Geotechnical Society 2006). Therefore there is expected to be some error in the values provided where load test results were adjusted to remove the unload and reload cycle. The 30 mm of toe displacement definition results in considerably greater capacities of up to 3,480 kN. This is 74% greater than the typical Q_{ULS} of 2,000 kN.

The key limitation in the analysis performed is assumptions required to calibrate the model. The *t-z* curves were estimated with an assumed beta of 0.25 which is in the typical range of clay but has not been verified locally. The *t-z* curve was also assumed to have elastic-perfectly plastic behaviour whereas the true behaviour could be strain-hardening or strain-softening. Lastly, residual forces in the pile prior to load testing were ignored. Despite these limitations, the analysis described in this paper provides an example for an improved methodology for calculating Q_{SLS} over current local practice.

Additional load testing on driven PPHC piles is recommended to better understand their performance and capacity in Winnipeg. Static load tests or bidirectional (Osterberg) cell tests on instrumented piles to measure the axial compression and strain distribution would assist in refining *t-z* and *Q-z* curves for the region. Dynamic testing is becoming more prevalent in Winnipeg and can provide insight into the variability of capacities across Winnipeg and related to driving methodology. A distribution of end bearing coefficients and shaft resistance can be established if a sufficient dataset becomes available. Publication of additional load testing results on driven PPHC piles in Winnipeg can aid local engineers in performing site specific capacity and settlement calculations.

Based on this exercise of performing a load-transfer analysis with calibrated *t-z* and *Q-z* functions from static load test results in Winnipeg, the following conclusions are drawn:

- The current typical practice can be conservative for both ULS and SLS design.
- The toe stiffness has a considerable effect on the serviceability.
- Serviceability is not likely to govern the design when the PPHC piles are adequately driven to a hard end bearing layer and the pile toe has a stiff response.

It is the authors' opinion that local practice in Winnipeg for the design of driven PPHC piles can be improved by considering site specific subsurface conditions. Performing separate capacity and settlement calculations better addresses the intent of ULS and SLS criteria rather than calibrating to historical allowable loads used in Winnipeg. The example calculations provided indicate there are opportunities to design more cost effective foundations. The potential economic benefits and construction savings for project owners should well outweigh the additional engineering effort and costs to perform a more thorough analysis on larger projects.

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