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Capacity of driven precast prestressed concrete hexagonal piles: a Winnipeg region geotechnical study

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ABSTRACT

Driven precast prestressed concrete hexagonal piles have been used to support industrial, commercial, and residential buildings in Winnipeg and the surrounding area since 1961. Pile design was originally governed by the National Building Code of Canada which stipulated the use of various methods, including static load tests, the Hiley dynamic pile driving formulae, and local experience. Governance of PPCH pile design was superseded in 1964 by the Metropolitan Corporation of Winnipeg By-law 711 which introduced maximum design capacities for specified pile sizes installed under ideal soil conditions. These capacities eventually became known locally as the “historical values” and provide the basis for the current state of local practice for geotechnical design of this pile type. The historical values were based on driving the piles to practical refusal in very dense silt till (hardpan). However, the historical capacities may not be appropriate where hardpan is absent and where sand inclusions under high porewater pressure may be present. The By-law was succeeded by the Manitoba Building Code in 1977 which omitted the historical values, although they are still commonly used today. This paper investigates the capacity of PPCH piles driven to practical refusal (estimated from dynamic pile load testing results) at three sites with varying till conditions to quantify potential differences between measured nominal capacities and those which would be derived from historical (allowable) capacities. The results demonstrate that there may be significant variability in the nominal capacity of PPCH piles driven to practical refusal in till indicating that the historical capacities may not account for this variability.

RÉSUMÉ

Depuis 1961, les immeubles résidentiels, commerciaux et industriels à Winnipeg et les alentours ont été soutenu par des pieux battu de béton précontraint hexagonaux. Initialement, le design de ce type de pieu était réglementé par le Code national du bâtiment du Canada. Ce dernier prévoyait une variété de méthodes pour le design, incluant les résultats sur l’essai de charges statiques, la formule dynamique du battage des pieux Hiley et la pratique locale. La gouvernance du design des pieux HBMP a été remplacer en 1964 par le règlement administratif 711 de la Metropolitan Corporation of Winnipeg qui a introduit des capacités de design maximum pour des tailles spécifiques de pieu installer dans des conditions du sol idéal. Éventuellement, ces capacités sont reconnues localement comme des ‘’données historiques’’ et donnent au usage local la base pour le design géotechnique pour ce type de pieu. Les données historiques étaient basées sur le refus d’un pieu d’une couche dense de silt till (calcin). Cependant, les capacités historiques pourront être appropriées ou non si le calcin est absent et l’endroit de l’inclusion de sable sous la haute pression sera possiblement présente. En 1977, le règlement administratif était suivi par le Code du bâtiment du Manitoba (CMB) qui omettait des données historiques, cependant elles sont encore utilisées aujourd’hui. Ce papier examine la capacité des pieux HBMP mobiliser sur le refus de pieu (estimer par les résultats dynamiques sur l’essai de charges) à trois sites de condition de till variée pour quantifier les différences potentielles entre les capacités nominales mesurer et celles qui seront remaniées de capacité historique (admissible). Les résultats démontrent qu’il pourrait avoir une variation significative de la capacité nominale du refus des pieux HBMP dans du till indiquant que ces capacités historiques ne peuvent pas tenir compte de cette variance.

1 INTRODUCTION

Driven precast prestressed concrete hexagonal (PPCH) piles have been extensively used to support industrial, commercial, and residential buildings in Winnipeg and the surrounding area since the 1960s. The current geotechnical design method for this pile type is based on a 1964 Winnipeg by-law which stipulated standard design capacities for specified PPCH pile sizes. These capacities are known as the historical values. Although the by-law was eventually succeeded by the Manitoba Building Code (MBC) a decade later, the historical values are still used today. Over time, it has become apparent that geotechnical design of PPCH piles has veered away from the main criterion upon which the historical values were based; that is, piles are to be driven to practical refusal in very dense silt till (i.e. hardpan). The historical capacities are commonly now applied to piles driven to refusal in all till or onto bedrock where the inherent variability is apparent and, in some cases, dictated by sand inclusions under high porewater pressure or the presence of cobbles and boulders.

Over the last 15 years and over the course of numerous projects, PPCH pile capacities have been measured by high-strain, non-destructive, dynamic load testing techniques. It is evident that mobilized pile capacity depends on driving criteria and the consistency and composition of the bearing soils. The information collected from dynamic load testing has led to a better understanding of load-transfer and development of PPCH pile capacity for correlation to the strength and composition of the bearing soils. Additionally, it has illustrated the effect that varying sub-surface conditions have on pile capacity, such as the presence of bedrock, cobbles or boulders and sand layers under high porewater pressure. This understanding is important considering the historical values are based on ideal soil conditions which are known to be absent in some areas of the region.

The primary purpose of this paper is to investigate the measured ultimate geotechnical capacity of PPCH piles driven to practical refusal in varying sub-surface conditions and compare these capacities with those derived from historical values. The investigation included a review of dynamic load testing results of PPCH piles from three sites in the Winnipeg region with different sub-surface conditions. A secondary purpose of this paper is to provide empirical data to aid in the selection of (locally) appropriate nominal capacities for calculation of the factored Ultimate Limit State (ULS) capacity.

This paper does not examine the load-displacement behavior of PPCH piles or long-term settlement associated with creep or driving-induced downdrag which are relevant for evaluation of Serviceability Limit State (SLS) capacity. Although not common practice in the Winnipeg region, static load tests are the most effective way to assess the load-displacement behavior of piles but are cost-prohibitive for most projects. The reader may refer to a recent study by Bartz and Blatz (2019) which investigates SLS capacity of driven PPCH piles.

2 GEOLOGICAL SETTING OF THE WINNIPEG REGION

The sub-surface stratigraphy in Winnipeg and the surrounding area generally consists of glaciolacustrine soils over till deposits underlain by sedimentary bedrock (predominantly dolomite or dolomitic limestone). Advancing glaciers during the last ice age deposited dense basal tills and glacio-fluvial sands and gravels over the scoured bedrock surface. When glaciers retreated north for the last time approximately 13,500 years ago, glacial Lake Agassiz was created (Teller, 1976) resulting in deposition of ablation (water-laid) tills followed by the glaciolacustrine clays and silts, with silty clay being the predominate layer.

The glaciolacustrine silty clay is generally highly plastic, weathered, heavily over-consolidated, brown (or mottled brown and grey) and stiff in the upper zone, becoming intermediate to high plastic, normally consolidated, grey, and firm to soft with depth. The clay is typically in the order of 10 m thick across the region but can be absent or up to 20 m thick. The undrained shear strength of the clay ranges from approximately 85 kPa in the upper brown clay to about 25 kPa in the lower grey clay (Kjartanson et al., 1983).

The till is comprised of a heterogenous mixture of clay, sand, gravel, cobbles, and boulders within a predominately silt matrix. The upper ablation tills are usually wet (water bearing) and loose, and the underlying basal tills are dry to moist and very dense to dense and are often referred to as hardpan. The till is encountered at depths ranging from ground surface to approximately 20 m within the Winnipeg region and ranges in thickness from 0 m to more than 20 m. Hardpan is generally known to consist of silt till with a water content in the range of 5% to 7% and a Standard Penetration Test (SPT) N-value greater than 50 blows/300 mm without influence from gravel size (or larger) particles. Saturated glacio-fluvial sand and gravel inclusions or layers are commonly present within the till deposits.

The underlying bedrock is sedimentary varying across the Winnipeg region from relatively weak shale and mudstones to higher strength carbonate, dolomite and dolomitic limestone. The shales and mudstones have unconfined compressive strengths (q_u) typically ranging from about 5 to 50 MPa, and the strongest dolomites and dolomitic limestones have q_u values in the order of 200 MPa. The depth to bedrock generally ranges from 15 to 20 m below ground surface with notable exceptions in the northwest part of the Winnipeg where the bedrock is near surface and in the southeast area where it is in excess of 30 m deep (Kjartanson et al.). The bedrock is typically fractured in the upper 1 m and contains sand, gravel, or till infilling. The carbonate rock is also karstic containing discontinuities such as fractures, joints, and caverns. Aquifers under high water pressure are present within the bedrock which can soften the till in areas due to the upward hydraulic gradient.

3 HISTORICAL DESIGN PRACTICE IN THE WINIPEG REGION

Driven PPCH piles are commonly used to support light to heavy industrial, commercial, and residential buildings. Manufacturing plants in Winnipeg make 300, 350, and 400 mm diameter piles which are available in lengths from 6 to 27 m. The specified pile cross-section dimensions are provided in Figure 1. Piles may be installed in large groups as shown in Figure 2 to support heavy loads for high-rise buildings or as single piles for floor slabs or single storey structures with light wall and column loads.

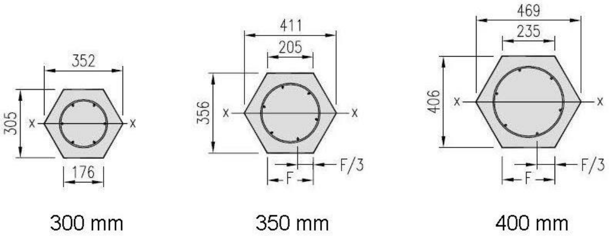

Figure 1: PPCH pile dimensions

Geotechnical design of PPCH piles was originally governed by the 1960 National Building Code of Canada (NBCC) based on the working stress design (WSD) method. The code stated that the capacity of a driven pile can be based on one of the following methods for design, including: 1) a load test carried out in accordance with good engineering practice, 2) local practice, 3) the end bearing capacity determined by the allowable soil bearing pressure, 4) the frictional capacity determined by the frictional resistance of the soil, or 5) the Hiley Pile Driving formula.

Figure 2: Large group of driven PPCH piles

Governance of PPCH pile design was superseded in 1964 by Metropolitan Corporation of Winnipeg By-law 711 which introduced maximum design capacities for the specified PPCH pile sizes installed in ideal soil conditions consisting of boulder-free hardpan. The By-law also stipulated that these capacities shall be reduced if significant thickness of hardpan does not exist above the bedrock or if a high frequency of boulders are known to exist within the till. These values became locally referred to as the historical or conventional capacities. The historical capacities provide valuable empirical information relative to the selection of appropriate nominal capacities for current Limit States Design methods. The pile capacities and driving criteria corresponding to the specified pile sizes are provided in Table 1.

Table 1. Manufactured PPCH pile sizes, historical allowable and ultimate pile capacities, and driving criteria

Pile Diameter (mm)[1]	Allowable Pile Capacity (kN)	Ultimate Pile Capacity (kN)[2]	Driving Resistance at Practical Refusal (blows / 25 mm)
300	450	1125	5
350	625	1565	8
400	800	2000	12

[1]distance between parallel sides of the pile

[2]based on an assumed factor of safety of 2.5

The By-law indicates that the historical capacities are based on an end bearing pile with no mention of shaft friction. Assuming a factor of safety of 2.5, the ultimate (nominal) unit end bearing resistance would be approximately 14,000 kPa.

According to the By-law, application of the historical values require that piles are driven “to virtual refusal in hardpan with hammers having sufficient energy to produce the desired results”. For piles weighing less than 3,410 kg (17 m in length or less), a hammer with a rated energy of 27 kJ should be used for installation and, for heavier piles, a hammer with a rated energy of 41 kJ should be used. In this regard, double acting diesel hammers (e.g. Link Belt I.C.E. 520) were typically used by local contractors. This hammer type has an energy transfer ratio (ETR) in the range of 20 to 80% (Tomlinson and Woodward, 2008). Based on the Author’s experience, double acting diesel hammers have an ETR of approximately 50%. It should be noted that hammer efficiencies have improved significantly since 1964.

By-law 711 was superseded by the MBC in 1977 which omitted the historical values from the design code. The MBC required that pile foundation design be based on the: 1) application of generally accepted geotechnical and civil engineering principles, 2) local practice, 3) a load test, or 4) innovative approaches. The MBC currently adopts the 2010 NBCC which states that design of foundations shall be based on 1) application of generally accepted geotechnical and civil engineering principles by a professional engineer especially qualified in this field of work, 2) established local practice, where such practice includes successful experience both with soils and rock of similar type and conditions and a foundation of similar type, construction method, size and depth, or 3) in-situ testing of foundation units, such as load testing of piles, carried out by a person competent in this field of work.

The maximum factored structural capacities of 300, 350, and 400 mm diameter piles are 1,075, 1,525, and 2,085 kN, respectively, based on short column analysis. These structural capacities are based on a concrete compressive strength (28-day) of 35 MPa for pile lengths of 26 m or less and 45 MPa for 27 m long piles as well as an approximate range of precompression stress due to prestressing of 2.8 to 7.3 MPa. For piles 26 m or less in length, the compression and tension limits for driving stresses range between 22 and 27 MPa and 4 and 9 MPa, respectively, depending on the magnitude of precompression due to prestressing. For 27 m long piles, the respective limits are approximately 31 to 35 MPa and 5 to 9 MPa.

3.1 Previous Pile Capacity Studies

A study, titled *The Manitoba Precast Prestressed Concrete Pile Capacity Study* (Conforce Ltd., Supercrete Ltd., Preset Piling, and Pile Foundations, 1978) was completed at the Con-Force Ltd. plant in Winnipeg in 1978 to ‘make available to designers and building officials a comprehensive statement of the “state of the art” with respect to the design of precast-prestressed hexagonal pile foundation in Manitoba’. The pile study included three independent studies comprised of static load tests, wave equation analyses, and pressuremeter testing. The main conclusion drawn was that PPCH piles driven into boulder-free hardpan consisting of a SPT N-value of 250 blows / 25 mm (or greater) can be designed with capacities 30% higher than the historical values using a corresponding increase in driving resistance, provided the following conditions are met: 1) a sub-surface investigation is completed to verify the quality of the bearing soils, 2) pile design is completed by a qualified person in the field of foundations and soil mechanics, and 3) pile installation is observed by a qualified inspector and the blow count and rebound is recorded at refusal. The ultimate end bearing resistance based on a 30% increase in capacity is estimated to be 18,750 kPa. Another study performed a few years later, titled *An Analysis of Pile Load Tests in the Winnipeg Area* (D.W. Spencer, 1982), which consisted of static load tests, suggested that the design capacities recommended in the 1978 study could be increased even further for piles installed in boulder-free hardpan or on bedrock.

4 CURRENT STATE OF PRACTICE IN THE WINNIPEG REGION

The historical capacities provided in By-law 711 form the basis of the well-established, current state of practice for geotechnical design of PPCH piles throughout the Winnipeg region. Although local practice now follows LSD methods, it is generally still rooted in the WSD method upon which the historical capacities were determined. The nominal geotechnical capacity of PPCH piles can be derived from the product of the allowable capacities and a factor of safety of 2.5. Some geotechnical practitioners may consider it appropriate to increase the nominal capacities derived by this method by 30% based on the results of the 1978 pile study. The factored Ultimate Limit State (ULS) capacity can then be calculated by applying a resistance factor. This approach to LSD has provided correlation to the conventional WSD method and has generally resulted in similar foundation designs.

With the emergence of dynamic load testing, there has been greater emphasis on driving piles to a final set with sufficient energy than the presence of hardpan. Dynamic load testing is now more frequently used to measure mobilized pile capacities, stresses and transferred energy, and assess pile integrity during driving. The relatively low cost and speed at which it can be performed (compared to static load tests), makes dynamic load testing a cost-effective tool to verify the design criteria. Dynamic load testing also allows for use of a higher resistance factor (0.5) according to the NBCC, potentially reducing the number of piles required.

The current state of practice for design and construction of PPCH piles generally consists of a geotechnical investigation during the design phase and inspection during construction. The geotechnical investigation typically consists of a sub-surface investigation to obtain samples for laboratory testing, assess the condition of the till and determine the depth of auger refusal. It may also be used to confirm bedrock. Care must be taken when interpreting auger refusal, especially if small diameter augers are used for sub-surface exploration (Skaftfeld, 2014). In some cases, auger refusal may be reached prematurely on cobbles or boulders rather than on hardpan which may be several metres deeper or absent. Large diameter augers (400 to 500 mm in diameter), such as used on a piling rig, are often used for foundation investigations as they can usually penetrate through cobbles and small boulders to reach hardpan (if present). It is also common to assume that PPCH piles will meet the historical driving criteria, and thus capacities, when driven to the depth of auger refusal. This has in many cases led to historical capacities being used for PPCH piles driven to refusal in till deposits absent of hardpan.

Inspection of PPCH piles is usually performed on a full-time basis during pile installation. Hammer drop height (or stroke) is monitored and penetration sets are directly marked on the pile during driving. Once three consecutive sets of the driving criteria are achieved at the proper energy (drop height or fuel setting) the pile is accepted from a geotechnical standpoint; hammer drop heights and fuel settings for common hydraulic drop and diesel hammers are well established in Winnipeg and generally accepted as delivering the required energy to the pile. Dynamic load testing may be implemented at the beginning of construction to verify pile capacity, hammer drop height (or fuel setting) and driving resistances (blow count), and to measure driving stresses. Confirmatory dynamic load testing may also be performed throughout foundation construction and to determine if pile set-up or relaxation has occurred.

The installation of driven PPCH piles in till can be challenging due to the variability of the till and drivability of PPCH piles. Cobbles and boulders, saturated sand and gravel inclusions, and weak zones within the till may cause pile damage, premature refusal, or piles not achieving the refusal criteria. Re-striking and dynamic load testing is often performed to verify pile capacity and integrity when these conditions are suspected or encountered.

5 DYNAMIC LOAD TESTING RESULTS FROM THREE SITES IN THE WINNIPEG REGION

A geotechnical study was completed at three sites (Sites 1, 2 and 3) within the Winnipeg region, as shown on Figure 3, to investigate the ultimate geotechnical capacity of PPCH piles driven to the historical driving criteria in variable sub-surface conditions and to compare the measured capacities with those derived from historical values. The study was based on the results of high-strain, dynamic load testing performed as a quality control measure during construction to verify pile design at each site. Mobilized end bearing and shaft friction resistances were also compared to the ultimate values upon which the historical capacities are based. Field and laboratory soil testing results, including SPTs, water content, and grain size distribution were reviewed, and the information was compared to typical values for hardpan.

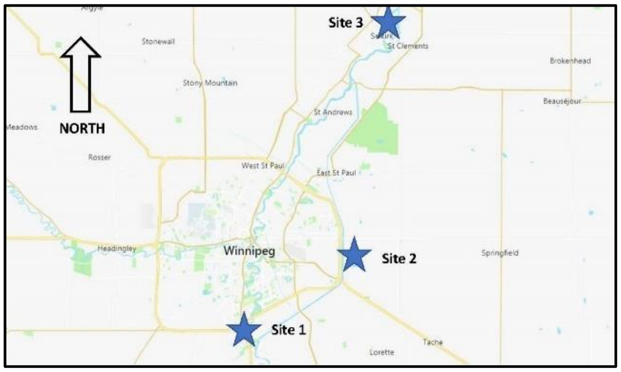

Figure 3: site locations (Bing Maps)

A Junttan HHK 5S external combustion, hydraulic drop hammer equipped with a 5,000 kg ram was used to install the PPCH piles at each site. This hammer type is commonly used to install PPCH piles in Winnipeg and has an energy transfer ratio (ETR) of 80 to 100% based on dynamic load testing data collected in region. Drop heights, typically used with this hammer, of 0.3, 0.35, and 0.4 m were used at end of initial drive (EOID) to drive 300, 350, and 400 mm piles to refusal respectively. In some cases, drop heights were increased during dynamic load testing in an attempt to mobilize additional capacity if driving stresses were within tolerable limits. Each pile was driven to three consecutive sets of the historical driving criteria corresponding to its size using the commonly used drop heights described in the upcoming section. Penetration sets for the prescribed number of blows were marked on each pile by hand using a carpenter's level and pencil and were measured using a ruler or a tape measure. Driving resistances reported in the following tables were recorded during EOID and prorated to 25 mm of pile penetration.

The dynamic load testing was performed using the Pile Driving Analyzer® (PDA) manufactured by Pile Dynamics Inc. (PDI) in accordance with ASTM D4945. Sensor measurements were collected using the PDA system and a representative blow was selected for signal-matching using the CAse Pile Wave Analysis Program® (CAPWAP) analysis software also developed by (PDI). The CAPWAP analyses are assumed to have followed proper practice in accordance with recommended procedures established by the developer of the CAPWAP software (PDI). The shaft and end-bearing resistances determined from the CAPWAP analyses were used for study. Quake, damping and other static and dynamic parameters used for the signal-matching analysis were not evaluated as part of this study

It was assumed that the ultimate geotechnical capacity of each pile was fully mobilized during testing and for the blows selected for signal matching. Each pile was tested under beginning of restrike (BOR) conditions ranging from 4 hours to 14 days after the end of initial drive (EOID).

The dynamic load testing results shown in this paper illustrate the variability in the end bearing and shaft resistances at the 3 sites. This variability can be attributed to variability of the till, the developed energy at the pile head, penetration set, and other factors which are not described in this paper (wait period between pile installation and time of testing and static and dynamic parameters used in the signal-matching process).

5.1 Site 1 – St. Norbert, South Winnipeg

Site 1 is in St. Norbert at the south end of Winnipeg (approximately 15 km south of downtown). The development is comprised of a 5-storey apartment complex supported by PPCH sizes designed based on the historical capacities and driving criteria.

Two test holes were drilled at the site to auger refusal within till at an average depth of 15.9 m below site grade using a DR-150 track-mounted drill rig equipped with 125 mm diameter solid stem augers. The soil stratigraphy encountered consists of 0.2 m to 1.7 m of fill materials over approximately 12 m of glaciolacustrine silty clay over till. Dolomite bedrock of the Red River Formation is known to be present below the site at a depth of about 18 to 21 m below ground surface (Kjartanson et al.).

The clay is generally stiff becoming very soft to firm with depth with S_u ranging from 75 kPa in the upper 5 m below ground surface to 20 kPa near the till surface. The till is comprised of some sand, some gravel, and trace clay within a silt matrix. The till is generally moist to wet and compact to dense. Water content measurements on till samples collected slightly above auger refusal depth vary between 13% and 14%. Standard Penetration Tests were performed in the till at the depth of auger refusal and N-values of 50 blows/12mm were recorded. No soil was recovered from the split spoons upon retrieval after completion the SPTs, indicating that auger and SPT refusal may have occurred on cobbles, boulders, or possibly bedrock. The till is not considered to be hardpan based on visual classification and water content.

All piles were driven to practical refusal from the base of an excavation approximately 2 m deeper than the ground surface from which the test holes were drilled. Dynamic load testing was completed on a 300, 350, and 400 mm diameter piles under BOR conditions 1 to 7 days after EOID. Pile details, transferred energy, and driving resistances are presented in Table 2 and mobilized capacities are shown in Table 3.

Table 2. Pile details, measured transferred energy, and driving resistance at EOID

Pile No.	Pile Diameter (mm)	Embedded Pile Length (m)	Maximum Delivered Energy (kJ)	Driving Resistance at Practical Refusal (blows/25mm)[1]
1	400	15.8	13	17
2	350	15.3	9	68[1]
3	300	16.7	4	76[1]

[1]converted from penetration sets per number of blows

Table 3. Mobilized pile capacities determined from dynamic load testing at BOR

Pile No.	Mobilized Capacity (kN)			Percent of Historical Nominal Capacity (%)
	End Bearing	Shaft Friction	Total	
1	1191	1239	2430	122
2	1400	697	2097	134
3	380	964	1344	120
			Average	125

The capacities of the three piles tested were on average 25% greater than the historical values. However, the results show that these piles are not entirely end bearing and in fact develop their capacity through a combination of end bearing and shaft friction resistance.

Shaft friction developed along the pile shafts accounted for 52% of the mobilized capacity with 41 kPa developed within the clay and 77 kPa within the till. End bearing resistance accounted for 48% of the mobilized capacity with an average end bearing resistance of 8,605 kPa, which is 61% of the historical ultimate value of 14,000 kPa. Assuming a geotechnical resistance factor of 0.5 (in accordance with the NBCC), the geotechnical capacities of the piles are less than the maximum factored structural capacities.

Average maximum compressive stresses measured during testing ranged from 14 to 18 MPa and average tensile stresses ranged from 1 to 2 MPa, which are well within the tolerable limits.

The ultimate capacity of the piles was possibly not fully mobilized during testing given the relatively small penetration sets and energy delivered to the piles. A higher drop could have been used in an attempt to mobilize more capacity considering the compressive driving stresses were well within tolerable limits.

The high blow count of Pile Nos. 2 and 3 at EOID and the pile embedment depths, compared to the anticipated depths to the bedrock surface (from geological maps of the area), indicates that the piles refused directly on bedrock or slightly above on boulders. However, this could not be confirmed based on the findings during the sub-surface investigation. Nonetheless, the piles did not reach refusal in hardpan.

5.2 Site 2 – RM of Springfield

Site 2 is within the RM of Springfield approximately 15 km east of downtown Winnipeg. The development includes a large heavily-loaded industrial building supported by 400 mm diameter PPCH piles designed based on the historical capacities and driving resistances.

Eleven test holes were drilled through the overburden soils and cored into bedrock using an Acker MP8 truck-mounted drill rig equipped with 125 mm diameter solid stem augers and HQ coring equipment. The soil stratigraphy at the site consists of approximately 3 m of fill materials (gravel over clay), 16 m of glaciolacustrine silty clay, 5 m of till and bedrock. The silty clay is highly plastic, and stiff to very stiff becoming very soft to firm with depth. Undrained shear strengths range between 40 to 110 kPa in the upper portion of the clay to about 15 to 40 kPa near the till interface. The till is comprised of varying amounts of clay, silt, sand, gravel, cobbles, and boulders and is generally moist to wet and loose to compact becoming moist to dry and dense with depth. Water content in the till ranges from 8 to 20% and SPTs N-values range from 13 to 61 blows/300mm. Several SPTs were terminated due to refusal on cobbles or boulders. The till is not considered to be hardpan.

All piles were driven from site grade to practical refusal in till and dynamic load testing was completed on nine 400 mm diameter piles under BOR conditions 2 to 14 days after EOID. Pile details, transferred energy, and driving resistances are presented in Table 4 and mobilized capacities are shown in Table 5.

Table 4. Pile details, measured transferred energy, and driving resistance at EOID

Pile No.	Pile Diameter (mm)	Embedded Pile Length (m)	Maximum Delivered Energy (kJ)	Driving Resistance at Practical Refusal (blows/25mm)[1]
1	400	22.0	14	13
2	400	24.8	28	29
3	400	21.2	24	15
4	400	21.8	24	16
5	400	21.8	13	16
6	400	21.6	19	13
7	400	25.0	25	19
8	400	21.8	23	14
9	400	24.5	21	13

[1] converted from penetration sets per number of blows

Table 5. Mobilized pile capacities determined from dynamic load testing at BOR

Pile No.	Mobilized Capacity (kN)			Percent of Historical Nominal Capacity (%)
	End Bearing	Shaft Friction	Total	
1	788	1639	2427	121
2	1972	1502	3474	174
3	704	1658	2362	118
4	1698	1451	3149	157
5	1211	1135	2346	117
6	1214	1282	2496	125
7	1026	1454	2480	124
8	1192	1122	2314	116
9	1374	1359	2733	137
			Average	132

The ultimate capacities of the nine piles tested exceeded the historical values by an average of 32%. Shaft friction accounted for 53% of the mobilized capacity with 22 kPa of shaft friction resistance developed within the clay and 111 kPa within the till. End bearing resistance accounted for 47% of the mobilized capacity. The piles had an average end bearing resistance of 8,701 kPa, which is 62% of the historical ultimate value of 14,000 kPa. The geotechnical capacities of the piles tested are less than the maximum factored structural capacities.

Maximum compressive stresses measured during testing ranged from 14 to 27 MPa with an average of 20 MPa. Higher drop heights were used during a few tests resulting in 44% more energy delivered to the piles and mobilization of 19% more capacity than piles driven using the typical drop heights at EOID. However, damage to one pile (not reported in the above table) prevented further implementation of this procedure. Maximum tensile stresses measured ranged from 2 to 4 MPa which are within tolerable limits.

5.3 Site 3 – Selkirk

Site 3 is located at the north end of the town of Selkirk approximately 35 km northeast of downtown Winnipeg. The development includes a large heavily-loaded industrial building supported by 350 and 400 mm diameter PPCH piles designed based on the historical capacities and driving resistances.

Eleven test holes were drilled to auger refusal within till at an average depth of 17.9 m using Acker Renegade and CME850 track-mounted drill rigs equipped with 125 mm diameter solid stem augers. The soil stratigraphy at the site consists of 10 m of glaciolacustrine silty clay overlying till. The silty clay is highly plastic, firm to stiff ($40<S_u<60$ kPa), becoming very soft to firm ($10<S_u<30$ kPa) with depth. The composition of the underlying till is highly variable. Grain size distribution varies from traces of clay, sand, gravel, cobbles and boulders within a silt matrix to clayey, sandy, gravelly, cobbly, and bouldery. The till is moist to wet and loose to dense. Water content measurements on till samples collected vary from 7 to 16% and SPT N-values range from 6 to 92 blows/300 mm, with several other SPTs refusing on coarse gravel, cobbles or boulders. Auger refusal is suspected to have occurred on cobbles or boulders. The till does not meet the Winnipeg criteria for hardpan based on water content and visual classification.

Sand and gravel layers (as thick as 3 m) under high porewater pressure were encountered within the till. Significant seepage and sloughing conditions were observed within these layers; in one test hole, over 10 m of base heave (i.e. blow-up) was measured and groundwater level rose as high as 3.7 m below grade upon penetration into a gravel layer. In some cases, soil samples could not be recovered from these layers. as measured immediately at completion of drilling.

Dynamic load testing was completed on two 350 mm piles and five 400 mm diameter sizes under BOR conditions 4 to 24 hours after EOID. Pile details, transferred energy, and driving resistances are presented in Table 6 and mobilized capacities are shown in Table 7.

Table 6. Pile details, measured transferred energy, and driving resistance at EOID

Pile No.	Pile Diameter (mm)	Embedded Pile Length (m)	Maximum Delivered Energy (kJ)	Driving Resistance at Practical Refusal (blows/25mm)[1]
1	350	16.6	19	8
2	350	16.5	23	15
3	400	16.6	12	16
4	400	17.2	36	13
5	400	16.7	33	15
6	400	18.4	42	20
7	400	16.4	28	15

[1] converted from penetration sets per number of blows

Table 7. Mobilized pile capacities determined from dynamic load testing at BOR

Pile No.	Mobilized Capacity (kN) End Bearing	Shaft Friction	Total	Percent of Historical Nominal Capacity (%)
1	528	488	1016	65
2	663	1166	1829	117
3	188	519	707	35
4	421	905	1326	66
5	677	762	1439	72
6	677	1133	1810	91
7	1760	926	2686	134
			Average	83

The seven piles tested had an average mobilized end bearing resistance of 5,277 kPa which is 38% of the historical ultimate value; the average end bearing value decreases to 3,714 kPa, or 27% of the historical values, if the two piles that met the historical capacities (Nos. 2 and 7) are omitted from the results. Shaft friction resistance accounted for 53% of the mobilized capacity based on 17 kPa of shaft friction resistance developed within the clay and 64 kPa within the till. Only two of the seven piles tested achieved the historical bearing capacities. The five piles that did not, had an average capacity of 66% of the historical values and one pile had an ultimate capacity less than the factored ULS capacity. Pile Nos. 2 and 7 almost certainly did not encounter the same soil conditions as the non-conforming piles and, as such, met the design criteria and achieve capacities 17 and 34% higher than the historical values.

Average maximum compressive stresses measured during testing ranged from 12 to 31 MPa, with two piles exceeding tolerable limits; however, no damage was indicated during testing or from the signal-matching results. The higher stresses were reflective of a higher drop height (energy) used to mobilize additional capacity. However, the capacity essentially remained unchanged from a typical drop height. Tensile stresses were in the order of 1 MPa and therefore within tolerable limits.

The cause of the low capacities at Site 3 was attributed to the presence of saturated sand and gravel inclusions. A buildup of porewater pressure occurred as piles were advanced through these layers which could not dissipate since the soils surrounding the inclusions are of relatively low permeability. The increased porewater pressure was assumed to cause significant damping effects and premature refusal. Where the conditions are known to exist or are suspected during pile installation, re-striking with dynamic load testing is commonly implemented.

5.4 A Closer look at 400 mm PPCH Piles

The ultimate capacities of the 400 mm PPCH piles at the three study sites are presented in graphical form in Figure 4 to illustrate the variability of the results and for comparison to the historical values.

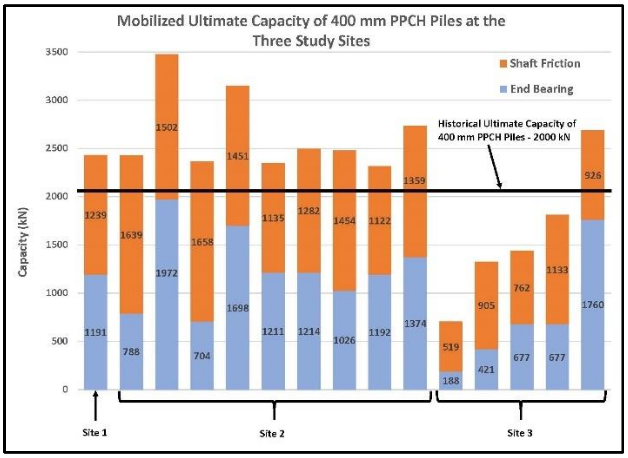

Figure 4. Capacity of 400 mm PPCH piles at the three study sites

The capacities at Sites 1 and 2 are between 16 and 74% higher than the historical values and on average 31% higher with an average ultimate capacity of 2,620 kN. These results are consistent with those presented in the 1978 pile study. At Site 3 however, the ultimate capacities are on average 34% lower than the historical values. A re-striking program was implemented to advance the piles at Site 3 until design capacities were achieved.

6 CONCLUSION

This study demonstrates the potential variability in the ultimate geotechnical capacity of PPCH piles driven to practical refusal in till. The results show that the presence of saturated sand and gravel layers can result in significantly less pile capacity than the historical values; this was evident at Site 3, where initial ultimate capacities were 34% lower than the historical values, requiring significant re-striking to overcome this phenomenon. Contrary to this, piles installed in till absent of these features can develop ultimate capacities 30% higher than the historical values, which is consistent with the results from the 1978 pile study and more recent studies by Bartz and Blatz (2019). Significantly higher ultimate capacities are likely achievable for PPCH piles installed in hardpan or bedrock since capacity would likely be governed by the amount of energy transferred to the pile, compressive driving stresses, and the strength of the concrete. The historical capacities do not account for this variability.

It appears that PPCH piles can achieve historical capacities even in the absence of hardpan provided they are driven to practical refusal using a hammer capable of delivering sufficient energy to the pile. The presence of adverse geological conditions within the till such as saturated pockets of sand and gravel can result in reduced capacities unless they are accounted for during driving or a re-striking program is implemented. The historical values were not intended to apply to piles installed in these conditions. For this reason, sub-surface investigations should be mindful of the potential presence of such adverse geological conditions within the till and should also include observation of groundwater conditions. Otherwise, reduced ultimate design capacities may be warranted.

The study results also indicate that the ultimate geotechnical capacity of driven PPCH piles installed to the historical driving criteria within the typical Winnipeg stratigraphy is derived from a combination of end bearing and shaft friction resistance, whether the piles refuse on bedrock or within till of varying conditions. The amount of frictional resistance is a function of the embedment depth of the pile and the strength the surrounding soils. Therefore, it is important to not only consider refusal depth for design but also the strength of the bearing soils along the pile shaft and at the pile toe. Correlation between soil testing (field and laboratory) results and pile resistances, in particular the end bearing resistance in various till conditions may be an important design consideration. Static load tests on piles equipped with strain gauges would be of benefit to verify load-transfer behavior and establish end bearing resistances in different clay and till conditions.

Dynamic load testing can be considered for projects that include driven PPCH piles to verify capacities, particularly when saturated sand and gravel layers or other adverse geological conditions are encountered in the till. The dynamic load testing results at Site 3 illustrate the importance of capacity verification during construction.

7 REFERENCES

Bartz, J. R. and Blatz, J. A. 2019. *A Modern Approach to the Design of Driven Precast-Prestressed Concrete Piles in Winnipeg*. Submitted for publication. GeoSt.John's 2019, Canadian Geotechnical Society, St. John’s, Newfoundland.

Building By-Law 711. 1964. Metropolitan Corporation of Winnipeg.

Conforce Ltd., Supercrete Ltd., Preset Piling, and Pile Foundations (1978). The Manitoba Precast Prestressed Concrete Pile Capacity Study.

Kjartanson, B., Baracos, A., and Shields, D.H. (1983). *Geological engineering report for urban development of Winnipeg*. Department of Geological Engineering, University of Manitoba, Print.

NBCC (2010). *National Building Code of Canada User's Guide - NBC 2010 Structural Commentaries* (Part 4 of Division B), NBCC, Ottawa, ON, CA.

Skaftfeld, K. 2014. *Experience as a Guide to Geotechnical Practice in Winnipeg*, M.Sc. thesis. University of Manitoba, Winnipeg, Manitoba

Spencer, D.W. 1982. *An Analysis of Pile Load Tests in the Winnipeg Area*. B.Sc. thesis, University of Manitoba.

Teller, J.T. 1976. *Lake Agassiz Deposits in the Main Offshore Basin of Southern Manitoba*, Canadian Journal of Earth Sciences, 13: 27-43

Tomlinson, M. and Woodward, J. 2008. *Pile Design and Construction Practice*, 5th Ed. Taylor and Francis, New York, New York, USA