# GEOTECHNICAL REPORT

# SEVEN OAKS POOL ADDITION
# 444 ADSUM DRIVE
# WINNIPEG, MANITOBA

Submitted to:

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# TABLE OF CONTENTS

## LIST OF TABLES



## LIST OF FIGURES



## LIST OF APPENDICES

# 1.0 INTRODUCTION

At the request of Mr. Evan Wiebe, C.E.T. and Project Officer for the Planning, Property & Development Department at the City of Winnipeg, Amec Foster Wheeler Environment & Infrastructure, a division of Amec Foster Wheeler Americas Limited (Amec Foster Wheeler), completed a geotechnical investigation for the proposed addition to Seven Oaks Pool located at 444 Adsum Drive, Winnipeg, Manitoba. The purpose of the geotechnical investigation was to investigate subsurface soil and groundwater conditions within the approximate footprint of the addition and to provide geotechnical recommendations for design. The scope of work for the project was outlined in Amec Foster Wheeler proposal WPG2015.573R, dated 9 January 2015. Authorization to proceed was granted under City of Winnipeg Purchase Order 388172.

This report summarizes the field and laboratory testing programs, describes the subsurface conditions encountered at the test hole locations, and presents geotechnical recommendations for the following: design and construction of applicable foundation alternatives; grade supported slabs; lateral earth pressures for basement walls; excavation and backfill; drainage and sub-drainage requirements; subsurface concrete requirement's; and testing and monitoring. The report also provides comments on unusual geotechnical conditions which may result in potential design or construction difficulties.

# 2.0 SITE AND PROJECT DESCRIPTION

## 2.1 Site Description

Seven Oaks Pool is an existing City of Winnipeg recreational pool located at civic address 444 Adsum Drive, Winnipeg, Manitoba. The location of the Site is illustrated in Figure 1.

At the time of the geotechnical investigation described in this report, existing site conditions within the footprint of the proposed addition predominantly comprised sloping landscape. The existing pool building is a single-storey pool building presumably supported on piles.

## 2.2 Project Description

Based on information provided with the request for proposal (RFP), Amec Foster Wheeler understood that the project consists of an approximate 50 m x 15 m addition to the north side of the existing pool building, starting approximately parallel with the west wall of the existing lobby entrance, and extending through to near the west perimeter wall of the existing structure. An approximately outlined of the proposed addition is shown in Figure 1.

Based on additional information provided by Wolfrom Engineering, structural agent for the City of Winnipeg, Amec Foster Wheeler understood that a shallow pool will occupy much of the western 28 m of the addition. It was understood that the pool will be approximately 800 mm deep at the west end shallowing up to about 150 mm for the majority of the middle area. It was also understood that the intent is to have a cast-in-place concrete tank with open crawlspace below; extending to as deep as 2 m below grade; to match conditions of the existing structure. The RFP indicated anticipated new pile loads in the range of 250 kN to 350 kN per pile.

# 3.0 GEOTECHNICAL INVESTIGATION PROGRAM

Prior to initiating drilling, Amec Foster Wheeler notified public utility providers (i.e. Manitoba Hydro, MTS, City of Winnipeg, etc.) of the intent to drill in order to clear public utilities, and where required, met with said representatives on-site. Amec Foster Wheeler also retained the services of a private utility locator to confirm clearance from privately owned utilities at the test hole locations.

On 25 and 28 February 2015, Amec Foster Wheeler supervised the drilling of a total of four test holes (TH01 through TH04) at the approximate locations illustrated in Figure 1. The holes were drilled to auger refusal in dense till between about 15.7 m and 16.8 m below grade using a track mounted DR-150 drill rig equipped with 125 mm diameter solid stem augers, operated by Maple Leaf Drilling Ltd. The location and elevation of the test holes were surveyed using Survey grade GPS with reference to Cansel's Can-Net Virtual Reference Station Network.

During drilling, AMEC field personnel visually classified the observed soils according to the Modified Unified Soil Classification System (MUSCS). Groundwater and drilling conditions were also recorded at the time of drilling. Grab samples were collected at selected depths from the auger cuttings, while relatively undisturbed Shelby tube samples were also collected between about 3.0 m and 9.0 m below grade in each of TH02 through TH04. The in-situ relative consistency of cohesive overburden was evaluated within the test holes using pocket penetrometer readings. The recorded pocket penetrometer readings are shown on the logs. The relative consistency of the underlying till was evaluated using a standard penetration test (SPT), where the number of blows to drive the SPT sampler 0.3 m into the soil was recorded. The recorded number of blows is shown on the logs as the SPT (N) value. Split spoon samples were also collected in conjunction with the SPT's.

The test holes were left open for approximately ten minutes after completion of drilling to observe the short-term groundwater seepage and sloughing conditions, and then backfilled with auger cuttings and a layer of bentonite at surface.

Following completion of the field drilling program, a laboratory testing program was conducted on selected soil samples obtained from the test holes. The laboratory testing program completed consisted of moisture content determinations and two unconfined compressive strength (UCS) tests completed in accordance with ASTM Standard D2166.

Detailed test hole logs summarizing the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole locations are presented in Appendix A. Actual depths noted on the test hole logs may vary by ± 0.3 m from those recorded due to the method by which the soil cuttings are returned to the surface. Summaries of the terms and symbols used on the test hole log and of the Modified Unified Soil Classification System are also presented in Appendix A.

# 4.0 SUBSURFACE CONDITIONS

## 4.1 Stratigraphy

Consistent with the regional geology and anticipated conditions, the stratigraphy at the test hole locations consisted of the following, in descending order from grade level:

- Topsoil (TH01), Gravel Fill (TH02 & TH03)
- Clay Fill
- Low Plastic Silt (TH01 and TH03)
- Clay (Lacustrine)
- Glacial Till (Silt and Clay)

Auger refusal within the four test holes occurred between approximate elevations 216.2 m (TH01, 16.2 m below grade) and 215.2 m (TH03, 16.8 m below grade).

Brief descriptions of each of the soil layers bulleted above is presented below. For detailed descriptions, the test hole logs in Appendix A should be consulted.

<u>Topsoil & Organic Clay</u>

Approximately 150 mm of topsoil and organic clay was encountered at the surface of test hole TH01 located within the landscape area along the existing Pool building. It should be noted that both the presence and the thickness of topsoil and organic clay across the footprint of the proposed addition will vary from that encountered at TH01.

<u>Gravel Fill</u>

Gravel fill was encountered at the surface of TH02 and TH03, and extended to approximately 1.5 m below grade at TH02, and about 0.15 m below grade at TH03. The gravel generally consisted of crushed limestone, and was described as sandy with some gravel, well graded, medium dense, dry, white to grey, and frozen.

<u>Clay Fill</u>

Clay fill was encountered beneath the organic clay at TH01; beneath the gravel fill at TH03; and at the surface of TH04. The clay fill was generally silty with trace sand, high plastic, damp to moist, firm to stiff, and brown to black. Furthermore, the clay fill was frozen to between 0.9 m and 1.2 m below grade. Trend analysis of all moisture contents within the clay fill generally indicated a relatively uniform moisture content profile at each test hole location, with values ranging from about 21 percent to 31 percent.

<u>Silt</u>

A thin layer of silt was encountered beneath the clay fill at TH01 and TH03 at shallow depths of about 1.5 m below grade (El. 230.9 m) and 0.3 m below grade (El. 231.7 m), respectively. The silt extended to depths of about 2.0 m below grade (El. 230.4 m) and 1.4 m below grade (El. 230.6 m), respectively. The silt was generally described as low to medium plastic with some clay to clayey, moist, firm, and tan. Moisture content results within the silt indicated an in-situ moisture content of about 20 to 21 percent. Pocket penetrometer index values of 50 kPa (TH03) and 75 kPa (TH01) were obtained.

Clay (Lacustrine)

Consistent with typical soil conditions in the Lake Agassiz Basin within which the Site is located, highly plastic lacustrine clay was encountered beneath the shallow silt layer at TH01 and TH03, as well as beneath the gravel fill and clay fill at TH02 and TH04, respectively. The clay extended to the underlying glacial till encountered between depths of about 13.7 m to 15.2 m below grade, or between approximate elevations 218.3 m to 216.7 m.

The clay was generally described as silty, highly plastic, moist to very moist becoming wet with depth, firm within the upper 4.5 m transitioning to soft with depth, and brown becoming grey with depth. Trend analysis of all moisture contents within the clay generally indicated relatively uniform moisture contents of about 50 percent to 62 percent, although a trend of decreasing moisture content with depth can be observed within TH01, TH02, and TH03 below approximate elevations 223.2 m, 219.4 m, and 221.4 m respectively. Overall, moisture content results within the clay ranged from about 32 percent to 62 percent. Pocket penetrometer index tests ranged from about 25 kPa to 175 kPa, with stiffer results observed in the upper 4.5 m to 9.0 m of grade.

Unconfined compressive strength tests were completed on two Shelby tube samples collected from TH02 and TH04; the results of which are summarized in Table 4-1.

**Table 4-1: Summary of Unconfined Compressive Strength Tests**

| Test Hole | Depth (m) | UCS (kPa) | Strain at 100% of UCS (%) | Strain at 50% of UCS (%) | Bulk Density ($kg/m^3$) | Dry Density ($kg/m^3$) |
|---|---|---|---|---|---|---|
| TH02 | 9.1 – 9.7 | 54 | 3.8 | 0.9 | 1773 | 1171 |
| TH04 | 3.0 – 3.6 | 64 | 10.3 | 1.8 | 1779 | 1153 |

Glacial Till (Silt and Clay)

Glacial till was encountered beneath the clay in all four test holes at depths of about 13.7 m to 15.2 m below grade, or between approximate elevations 218.3 m to 216.7 m. The till extended to auger refusal at depths of about 0.8 m below grade to 3.0 m below the top of the till, or between approximate elevations 216.2 m and 215.2 m.

The till generally comprised a low plastic silt and clay matrix with trace to some sand and some gravel becoming gravelly. Moisture conditions within the till varied from damp to wet, with in-situ moisture contents within the till ranging from as little as about 8 percent at auger refusal in TH04 to as much as about 41 percent in the upper 1.5 m of till at TH03. The relative density of the till ranged from soft near the clay/till interface to hard at auger refusal. A single SPT N value of 14 was measured at TH04 approximately 1.2 m below the surface of the till. SPT values in TH01, TH02, and TH03 all refused prior to achieving the standard penetration of 450 mm (i.e. 3 sets of 150 mm). Equivalent SPT penetration resistances of 2.4 mm to 3.0 mm per blow (i.e. SPT ‘N’ values of 100 to 125 per 300 mm of penetration) were noted where SPT refusal was observed.

Generally, soft conditions are expected near the clay interface, the moisture content of the till is expected to decrease with depth, and the stiffness/density of the till is expected to increase with depth.

## 4.2 Groundwater and Sloughing Conditions

Seepage and sloughing conditions were noted during drilling, and the depth to the accumulated water level within the test holes was measured about ten minutes after drilling at each test hole location. Sloughing during drilling was observed within the glacial till in each of the deep test holes. The depths to slough and groundwater noted during drilling and upon drilling completion are summarized in Table 4-2.

**Table 4-2: Observed Slough and Groundwater Conditions**

| Test hole | Depth Explored (m) | During Drilling | | Upon Completion | |
|---|---|---|---|---|---|
| | | Depth(s) of Slough (m) | Depth(s) of Seepage (m) | Depth to Slough (m) | Depth to Groundwater (m) |
| TH01 | 16.2 | None | Below 14.6 | 15.5 | 11.3 (El. 221.1 m) |
| TH02 | 15.7 | Below 12.8 | Below 15.2 | 7.9 | 7.9 (El. 223.7 m) |
| TH03 | 16.8 | Below 13.7 | Below 12.2 | 14.6 | 11.0 (El.221.0 m) |
| TH04 | 16.6 | Below 12.2 | Below 15.2 | 16.2 | 9.4 (El.225.5 m) |

It should be noted that only short-term seepage and sloughing conditions were observed and that groundwater levels can fluctuate annually, seasonally or as a result of construction activity. Groundwater conditions in the region of the Site are generally considered to be controlled by artesian conditions within the underlying till and bedrock. The groundwater table at the site is estimated at approximate elevation 227 m.

# 5.0 GEOTECHNICAL RECOMMENDATIONS

## 5.1 General Evaluation

The stratigraphy and soil conditions encountered within the test holes advanced at the site are considered typical of conditions within the area of the Site (i.e. within the Lake Agassiz Basin) and are considered favourable for the proposed development. From a foundations perspective, soil conditions are considered suitable for the use of a variety of pile foundation alternatives including driven pre-cast pre-stressed concrete (PPC) piles and conventionally bored straight shaft cast-in-place (CIP) concrete piles. Based on discussions with Wolfrom Engineering, structural design consultant for the project, Amec Foster Wheeler understood that driven PPC piles comprise the preferred foundation alternative for support of the single pile loads outlined in Section 2.2. Amec Foster Wheeler supports this proposed foundation type, and in this regard, foundation recommendations in this report are limited to the preferred PPC piles, as well as conventionally bored CIP friction piles for consideration in support of light foundation loads. End bearing CIP concrete piles extending into the underlying till are not recommended given the artesian groundwater pressure in the till noted upon drilling completion. Recommendations for alternative foundations can be provided upon request.

With respect to subgrade conditions for grade supported concrete floor slabs, the shallow silt layer encountered at two borehole locations between approximate elevations El. 230.9 m and El. 231.7 m and extending to between approximate elevations El. 230.4 m and El. 230.6 m presents a risk for soft subgrade conditions at shallow depth. Furthermore, assuming final site grades similar to existing, the shallow silt layer is located within the depth of seasonal frost penetration. Moist to wet silt such as that encountered at the site is highly frost susceptible in comparison to clay, and in this regard, the shallow silt presents a greater risk for frost heave of grade supported structures (if applicable) in unheated areas. Additional discussion of subgrade conditions for grade supported slabs is presented in Section 5.8.1.

The following sections provide discussion and recommendations as they pertain to: driven pre-cast pre-stressed concrete piles; bored concrete piles; frost design considerations; grade supported concrete slabs; asphalt pavement; final site grading and drainage; and foundation concrete.

## 5.2 Driven Pre-cast Pre-stressed Concrete Piles

### 5.2.1 Axial Compressive Resistance

Hexagonal pre-cast pre-stressed concrete (PPC) piles driven to practical refusal in the very dense till under an appropriate hammer energy and force configuration are considered a suitable pile foundation alternative. Based on the depth to auger refusal during drilling, it is anticipated that pre-cast concrete piles could be driven to practical refusal at the site between about 15.0 m and 17.0 m below existing grade, although it should be noted that shallower or deeper pile refusal may also occur depending on undulations in the dense glacial till surface and variability in the relative density of the till.

The unfactored (ultimate) resistance of a driven pre-cast concrete pile should be limited on the basis of the pile properties and the hammer used to drive the piles. Typically, a minimum hammer energy of 40 kJ per blow is recommended in driving pre-cast pre-stressed concrete piles to practical refusal in Winnipeg. To reduce the potential for structural damage to the piles, the piles should not be driven beyond practical refusal as noted in Table 5-1. The preliminary unfactored (ultimate) resistance should be limited based on hexagonal pile diameter as outlined in Table 5-1.

**Table 5-1: Single Pile Resistance Limits for Pre-cast Concrete Piles - ULS**

| Hexagonal Pile Size (mm) | Ultimate Pile Capacity (kN) | Refusal Criteria (blows/25mm) | |
|---|---|---|---|
| | | No Follower | With Steel Follower |
| 300 | 1,350 | 5 | 6 |
| 350 | 1,650 | 8 | 10 |
| 400 | 2,100 | 12 | 15 |

Based on the *2010 National Building Code of Canada* (NBCC 2010), a geotechnical resistance factor, $\Phi = 0.4$ should be applied to the ultimate geotechnical compressive resistance of the pile to obtain the factored geotechnical resistance at the Ultimate Limit State (ULS) for compressive loading conditions.

Additional comments for design and construction of driven pre-cast concrete piles are as follows:

- Pre-cast concrete piles typically require pre-bore pilot holes to facilitate pile installation. Pre-boring to a maximum depth of about 1/3 of the pile length (i.e. 5 m below grade) is recommended to promote pile verticality and alignment, and to reduce the effects of pile heave during driving of adjacent piles, which is particularly important in pile groups. The recommended pre-bore depth should be strictly controlled and monitored during construction, as large diameter and deep pre-bores could provide a conduit for upward seepage of groundwater from the underlying till and bedrock. Common to local construction practice, a pre-bore diameter slightly larger than the circumscribed circle of a hexagonal pile is employed. However, where shaft friction is required over the pre-bore depth and/or where lateral support is critical, pre-bore holes should be limited to no larger than 85% of the nominal pile diameter such that the pre-cast piles fit tightly in the drilled holes.
- The pile cross sections must be designed to withstand the design loads and the driving forces during installation.
- Frost design considerations are outlined in Section 5.6.
- Recommendations for uplift resistance calculations are provided in Section 5.2.2.
- Piles must be spaced a minimum of three pile diameters apart, as measured from centre-to-centre, in order to act individually as single piles in vertical compression when used in a small pile group of three piles or less. Where larger pile groups are required, the pile group should be reviewed by Amec Foster Wheeler.
- All piles driven within five pile diameters should be monitored for heave and, where heave is observed, piles should be re-driven. Piles that are re-driven should be advanced to at least the original elevation, and to the required termination criteria.
- All piles should be driven continuously to practical refusal once driving is initiated.
- Any piles that have been damaged, are excessively out of plumb, or have refused prematurely may need to be replaced, pending a review by a qualified geotechnical engineer of their load carrying capability and estimated settlement.
- All pile caps and grade beams should be underlain by a minimum 150 mm thick void form to accommodate the expansive nature and potential frost heave of the underlying soil.
- Monitoring of the pile installations by qualified personnel is recommended to verify that the piles are installed in accordance with design assumptions and that driving criteria are satisfied.

It should be noted that these recommendations pertain to geotechnical resistance only, and that the factored resistance for a pile shall be taken as the lesser of the factored geotechnical resistance or the factored structural resistance of the pile.

### 5.2.2 Tensile (Uplift) Resistance – Driven PPCP

In the case of driven straight shaft PPCPs, the uplift resistance of a single pile will be provided by the sustained downward load on the pile (if applicable) and shaft friction along the length of pile embedded below the depth of frost penetration. The unfactored (ultimate) uplift resistance of a driven pre-cast pre-stressed concrete pile due to shaft friction can be determined using the unfactored unit shaft friction values outlined in Table 5-2; however the length of pile over which shaft friction can be included in calculation of uplift resistance will depend on the diameter and depth of the pre-bore employed during construction. Where the pre-bore is larger than the inscribed circle of a hexagonal pile, shaft friction must be neglected over the full depth of the pre-bore. Where the pre-bore is equal to or smaller than the inscribed circle of the hexagonal pile, shaft friction may be applied over the length of pile below the depth of frost.

**Table 5-2: Unit Shaft Friction for Uplift Resistance of PPCP - ULS**

| Design Elevation[1] (m) | Assumed Soil Type | Unfactored Unit Shaft Friction (kPa) |
|---|---|---|
| Above El. 232.0 | New Fill | 0 |
| Final Grade Elev. to X m Below Final Grade Elev.[2] | Fill and Stiff Clay | 0[2] |
| X m Below Final Grade Elev.[2] to El. 227.0 m | Stiff to Firm Clay | 40 |
| El. 227.0 m to El. 216.0 m | Firm to Soft Clay | 25 |

[1] Existing grade has been nominally assumed equal to elevation 232.0 m.
[2] X = 1.5 m below slab/crawlspace grade in heated areas, or the depth of frost penetration in unheated areas, as recommended to account for possible movement of the soil away from the perimeter of the pile. If the pre-bore is larger than the inscribed diameter of the pile, then X shall be taken as the greater of the aforementioned and the depth of the pre-bore.

Based on the *2010 National Building Code of Canada* (NBCC 2010), a geotechnical resistance factor, $\Phi = 0.3$ should be applied to the unfactored geotechnical tensile resistance of the pile to obtain the factored geotechnical resistance at the Ultimate Limit State (ULS) for tensile loading conditions.

### 5.2.3 Serviceability and Pile Settlement – Driven PPCP

The settlement of a single pile depends on the applied load, strength-deformation properties of the foundation soils, load transfer mechanism, load distribution over the pile embedment depth, and the relative proportions of the load carried by shaft friction and end-bearing. A pile settlement limit value was not specified by the structural agent for use in developing geotechnical resistance limits for the serviceability limit state design criterion. Notwithstanding, assuming good workmanship, inclusive of good excavation, the predicted settlement of a PPCP at *working loads equal to a maximum given by 40 to 50 percent of the ultimate resistance of the pile* is 0.5 to 1 percent of the shaft diameter plus the elastic shortening of the pile due to the compressive load acting on the pile.

### 5.2.4 Lateral Resistance (Single Pile)

Significant horizontal (or lateral) loading conditions requiring evaluation of lateral load resistance of piles is not anticipated. Consequently, recommendations pertaining to the lateral load resistance of piles are not provided here-in. AMEC can undertake evaluation of lateral pile capacity if required, upon request.

## 5.3 Bored Concrete Piles

### 5.3.1 Axial Compressive Resistance

The consistency of the lacustrine clay overburden at the site is considered suitable for the support of light to moderate foundation loads on bored straight shaft cast-in-place concrete friction piles embedded above the till. The unfactored (i.e. ultimate) compressive resistance of a bored straight shaft CIP friction pile can be determined using the ultimate shaft friction values presented in Table 5-3. Given measured groundwater levels above the till surface within 10 minutes of drilling completion, the potential for seepage and basal softening of shafts extending into the till is considered high. In this regard, extension of the drilled shafts piles into the till is not recommended. If an end bearing pile design is to be provided, driven PPC piles are advised.

**Table 5-3: Unit Shaft Friction for Uplift Resistance of PPCP - ULS**

| Design Elevation[1] (m) | Assumed Soil Type | Unfactored Unit Shaft Friction (kPa) |
|---|---|---|
| Above El. 232.0 | New Fill | 0 |
| Final Grade Elev. to X m Below Final Grade Elev.[2] | Fill and Stiff Clay | 0[2] |
| X m Below Final Grade Elev.[2] to El. 227.0 m | Stiff to Firm Clay | 40 |
| El. 227.0 m to El. 219.0 m | Firm to Soft Clay | 25 |

[1] Existing grade has been nominally assumed equal to elevation 232.0 m.
[2] X = 1.5 m below slab/crawlspace grade in heated areas, or the depth of frost penetration in unheated areas, as recommended to account for possible movement of the soil away from the perimeter of the pile. If the pre-bore is larger than the inscribed diameter of the pile, then X shall be taken as the greater of the aforementioned and the depth of the pre-bore.

Based on the *2005 National Building Code of Canada* (NBCC 2005), a geotechnical resistance factor, $\Phi = 0.4$ should be applied to the unfactored geotechnical compressive resistance of the pile to obtain the factored geotechnical resistance at the Ultimate Limit State (ULS) for compressive loading conditions. The following recommendations also apply to the design of bored cast-in-place concrete piles.

- The weight of the embedded portion of the pile may be neglected in the design.

- The pile embedment depth, pile diameter, steel reinforcement and concrete compressive strength should be determined by the structural engineer, as required, to provide sufficient resistance to the applied loads.
- For conventionally bored straight shaft piles, the minimum pile spacing should be at least 2.5 pile diameters. The excavation of adjacent piles within three (3) pile diameters should be deferred until the concrete in the constructed pile has set.
- Frost design considerations are outlined in Section 5.6.
- Recommendations for uplift resistance calculations are provided in Section 5.2.2.
- A void space (minimum of 150 mm thick) should be constructed, using a compressible and biodegradable material, below all piles caps and to accommodate movements of the underlying soil.

Recommended procedures for the installation of conventionally bored, cast in-place concrete piles are:

- Slight seepage from the silt layer and 'necking' or squeezing of the test hole beyond 9.1 m below grade (i.e. below groundwater table) were noted during drilling. Should sloughing soil conditions and/or water bearing silt or sand layers be encountered during pile installation, steel casing should be installed in the augured excavations to control caving and groundwater seepage so that piles are cast in clean, dry holes. The level of fresh concrete in the casing must be maintained above the caving or seepage zone as the casing is withdrawn, and should be sufficiently high to equilibrate pressures inside and exterior of the casing to prevent collapse or squeezing of the sidewall into the pile bore.
- All piles should be poured immediately after completion of drilling to reduce the potential for seepage and swelling or squeezing of the pile bore, as well as to mitigate stress relief which could negatively impact pile settlement performance. Concrete should be poured in accordance with the latest edition of Canadian Standards Association A23.1 (Concrete Materials and Methods of Concrete Construction). Where required, dewatering of pile test holes should be managed using a bailing bucket or a submersible pump subject to actual field conditions.
- A qualified and experienced inspector should be on site during the entire period of pile installation. The inspector should keep complete and accurate records of the pile installations. For belled piles, the pile inspector should verify that a competent bearing stratum has been attained and that the base of the bell has been adequately cleaned.

### 5.3.2 Tensile (Uplift) Resistance

Piles resisting structural uplift loads and single pile structures (such as light poles) will need to be designed to resist tensile loads induced by frost and transient loads. Frost design penetration depths and design considerations are outlined in Section 5.6.

In the case of CIP straight shaft fiction piles, the uplift resistance of a single pile will be provided by the sustained downward load on the pile (if applicable) and shaft friction along the length of pile embedded below the depth of frost penetration. The unfactored (ultimate) uplift resistance of CIP concrete piles due to shaft friction can be determined using the unfactored unit shaft friction values outlined inTable 5-2.

Based on the *2005 National Building Code of Canada* (NBCC 2005), a geotechnical resistance factor, $\Phi$ = 0.3 should be applied to the unfactored geotechnical tensile resistance of the pile to obtain the factored geotechnical resistance at the Ultimate Limit State (ULS) for tensile loading conditions.

#### 5.3.3 Serviceability and Pile Settlement

The settlement of a single pile depends on the applied load, strength-deformation properties of the foundation soils, load transfer mechanism, load distribution over the pile embedment depth, and the relative proportions of the load carried by shaft friction and end-bearing. A pile settlement limit value was not specified by the structural agent for use in developing geotechnical resistance limits for the serviceability limit state design criterion. Notwithstanding, assuming good workmanship, inclusive of good excavation, the predicted settlement of CIP concrete piles at *working loads equal to a maximum given by 40 percent of the ultimate resistance of the pile* are:

- For bored straight-shaft fully friction-type concrete piles where end-bearing is neglected, the predicted settlement of a single pile would be in the range of 0.1 to 0.5 percent of the shaft diameter, plus elastic shortening due to the compressive load acting on the pile.

#### 5.3.4 Lateral Resistance (Single Pile)

Significant horizontal (or lateral) loading conditions requiring evaluation of lateral load resistance of piles is not anticipated. Consequently, recommendations pertaining to the lateral load resistance of piles are not provided here-in. AMEC can undertake evaluation of lateral pile capacity if required, upon request.

### 5.4 Pile Group Effects

AMEC does not anticipate that foundation loads will necessitate large groups of four or more closely spaced piles. Consequently, recommendations pertaining to the axial and lateral load resistances of pile groups are not provided here-in. If pile groups are required by design, AMEC should be notified and a review of possible group interactions effects evaluated.

### 5.5 Downdrag, "Drag Load", and Negative Shaft Friction

Downdrag, "drag load" and negative shaft friction are phenomena relating to the relative movement between the soil and pile at the soil/pile interface. They are of particular concern to pile settlement and/or over-stressing of a pile section when earthworks or surface loads at the site result in a significant change to the stress state of the soil surrounding the foundation. Such is the typically the case when significant fill is placed at a site to raise existing grade.

Amec Foster Wheeler has assumed that any raise in site grades will be limited to 300 mm or less above existing grade. In this regard, downdrag, “drag load”, and negative shaft friction are not of particular concern. If site grades are to be raised by more than 300 mm, then AMEC should be contact for review of potential development of downdrag and “drag load” of the pile foundation(s).

## 5.6 Frost Design Considerations

### 5.6.1 Frost Penetration Depth

The upper stratigraphy at the test hole locations, and across the site, is considered moderately to highly frost susceptible in the presence of water, and as such, frost effects should be considered for foundations or surface structures sensitive to movement. Based on historical temperature data for the area, a design frost penetration, assuming cohesive soils from ground surface, may be taken as 2.4 m below final grade in unheated areas that will not have regular snow or vegetative ground cover. Where there is beneficial heat loss into the soil from the superstructure and/or foundations, the depth of frost penetration may be as low as 1.5 m along the perimeter of the structure, subject to the details of the structure. Alternatively, the depth of frost penetration (and thus frost effects) may potentially be reduced by installing insulation. AMEC can provide recommended insulation details for specific development conditions upon request.

### 5.6.2 Pile Foundations

Frost forces applied to pile foundations include adfreeze pressures acting along the pile shafts within the depth of frost penetration. If pile caps are used and extend beyond the perimeter of the underlying pile, then frost heave forces acting on the undersides of the pile caps, as well as any connecting supports (i.e. lateral tie between the piles) will also need to be considered.

#### *5.6.2.1. Frost Heave*

To reduce the potential of frost heave pressures, a void-forming product should be installed beneath the underside of the pile caps and any other structural element located within the depth of frost penetration. The recommended minimum thickness of the void should be 150 mm. Alternatively, a compressible material may be used in lieu of a void forming material, and the uplift pressures may be taken as the crushing strength of the compressible medium. It is recommended that a frost heave of 150 mm be assumed in determining the required thickness for the void-filler and the associated uplift pressures associated with the thickness used.

The finished grade adjacent to each pile cap or grade beam should be capped with well compacted clay and sloped away so that the surface runoff is not allowed to infiltrate and collect in the void space or in the compressible medium.

#### *5.6.2.2. Adfreeze Stresses*

Resistance to adfreeze and frost heave forces will be provided by the sustained vertical loads on the foundation, the buoyant weight of the foundation and dead weight of the structure, and the soil uplift resistance component provided by the length of the pile extending below the depth of frost penetration. In the case of straight shaft piles supporting lightly loaded unheated facilities, the piles should be embedded a minimum of 8 m below final grade in order to provide sufficient frictional resistance against potential adfreeze stresses. For heated structures which allow beneficial heat loss into the soil, minimum pile lengths of 6 m are recommended. Where piles for heated structures are exposed to unheated conditions during construction, they should be designed for the unheated condition.

Adfreeze stresses along the sides of pile caps and buried substructures can be reduced by the installation of a 'bond-break' or 'friction reducer' within the zone of frost penetration. Friction reducers could consist of a system of poly wrapped sono-tubes. A smooth geosynthetic liner material, fixed to the shaft of the pile or to the sides of the pile cap would also be a suitable bond-break.

## 5.7 Lateral Earth Pressures on Below Grade Walls

### 5.7.1 Lateral Earth Pressure Coefficients and Soil Parameters

Grade beams and walls extending below grade will be required to resist lateral pressures from the surrounding soil, water, and any additional surcharge loading (i.e. fill, live surface loads, etc.). Recommended earth pressure distributions for moderate to well compacted backfill cases, as well as for line or point surcharge loads, are presented in Section 5.7.2. Table 5-4 provides recommended design values for the bulk unit weight, angle of internal friction, and lateral earth pressure coefficients for moderately to well compacted common fill (Clay) expected to be employed in backfill of excavations for grade beams and basement walls. Given that it is anticipated that grade beams and foundation walls will be designed to be 'un-yielding', only lateral earth pressures for the "at-rest" condition (i.e. $K_o$) are provided.

**Table 5-4: Earth Pressure Coefficients and Soil Unit Weights**

| Soil Type | | "At Rest" Earth Pressure Coefficient $K_o^*$ | Total Soil Unit Weight (kN/m$^3$) | Friction Angle (deg) Between Soil and Concrete |
|---|---|---|---|---|
| Common Fill (Clay) | Well Compacted (i.e.>=98% SPMDD) | 0.69* | 18 | 18 |
| | Moderately Compacted (i.e. 95 to <98% SPMDD) | 0.74* | 16 | 15 |
| Gravel Fill | Well Compacted | 0.40* | 23 | 25 |
| | Moderately Compacted | 0.47* | 22 | 21 |
| Sand Fill | Well Compacted | 0.47* | 21 | 21 |
| | Moderately Compacted | 0.53* | 20 | 18 |

* In the case of unyielding walls exposed to frost penetration above the groundwater table, it is recommended that $K_o$ = 1.0, be used to account for lateral frost pressures[1].

[1] As per Canadian Foundation Engineering Manual, 3rd Edition, P. 429.

With respect to subsurface drainage and groundwater conditions over the depth of the wall, the use of free draining backfill and the provision of drainage behind vertical subsurface walls are recommended, and will serve to mitigate groundwater accumulation and frost action on vertical walls extending through the zone of frost penetration. Where sub-drainage will not be provided behind a wall, buoyant soil unit weights should be used below the water table, and a hydrostatic pressure component will need to be included in the design. Buoyant soil unit weights are determined by subtracting the unit weight of water (10 kN/m$^3$) from the total unit weights provided in Table 5-4. The design groundwater table selected for design should consider the possibility for elevated or perched groundwater conditions resulting from infiltration within the backfill.

### 5.7.2 Calculation of Earth Pressure Distributions and Load Factors

The magnitude and distribution of the lateral earth pressures on below grade structures will depend on such factors as the rigidity of the below grade structure; the degree of compaction of the backfill against the structure; the backfill soil type; the slope angle at the structure/soil interface; and the subsurface drainage and groundwater conditions over the height of the structure. In addition to earth pressures, lateral stresses generated by any applicable surcharge loads also need to be evaluated in the design. The following subsections present recommended earth pressure distributions for moderate to well compacted backfill cases; as is recommended as the minimum level of compaction on bridge approaches; as well present pressure distribution for line or point surcharge loads.

### 5.7.3 Moderate to Well Compacted Backfill Case

Where subgrade support on the surface of the retained soil behind a wall is required, the backfill against the wall will need to be compacted to at least 95 percent Standard Proctor maximum dry density. The use of free draining backfill behind below grade structures is recommended in order to maintain drained conditions behind the structure. Assuming drained conditions, the design earth pressure distribution should adopt a combined trapezoidal/triangular distribution as shown on Figure 2 to account for the induced lateral pressures due to compaction. Figure 2 also provides the relationships to be used in the calculation of the compaction induced earth pressures, and tabulated loads (P) generated by typical compactors. The earth pressure coefficients to be used in the calculation of the lateral pressures should be those applicable to the backfill types given in Table 5-4.

If sub-drainage is not provided and it is possible by design for a perched groundwater to develop within the retained soil (i.e. “bathtub” effect associated with gravel fill soils surrounded by low permeable fine grained soil types), the hydrostatic component should be included in addition to the earth pressure given in Figure 2.

### 5.7.4 Surcharge Loads

In addition to earth pressures, lateral stresses generated by surcharge loads, such as loads from vehicles or grade supported structures along the perimeter of the addition (if applicable), also need to be evaluated in the design. For line or point surcharge loads, the lateral pressures should be

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combination with insulation for highly frost susceptible soils.

determined using the relationships given in Figure 3. In the case of uniformly distributed surcharge loads, such as those acting on the surface of the retained soil, the induced lateral earth pressure may be determined by multiplying the surcharge load by the appropriate earth pressure coefficient.

## 5.8 Grade-Supported Concrete Slabs

### 5.8.1 General

With respect to subgrade conditions for grade supported concrete floor slabs, the shallow silt layer encountered at two borehole locations between approximate elevations El. 230.9 m and El. 231.7 m and extending to between approximate elevations El. 230.4 m and El. 230.6 m presents a risk for soft subgrade conditions at shallow depth, and corresponding construction difficulties. Furthermore, assuming final exterior site grades similar to existing, the shallow silt layer is located within the depth of seasonal frost penetration. Moist to wet silt such as that encountered at the site is highly frost susceptible in comparison to clay, and in this regard, the shallow silt presents a greater risk for frost heave of grade supported slabs in unheated areas (i.e. sidewalk slabs).

Further to the shallow silt and the in-situ moisture condition of the clay the volumetric change potential (i.e. swell and shrinkage) of the clay in response to changes in soil moisture is qualitatively high. This is characteristic of the montmorillonite mineralogy of the highly plastic lacustrine clays within the Lake Agassiz Basin. In this regard, moisture conditioning, protection of the subgrade during construction, and final site grading and surface drainage will be important in managing the risks associated with volumetric change of the subgrade soils beneath foundation elements and grade supported structures/slabs in response to drying and/or wetting. Clay soils with moisture contents in excess of 50 percent are currently susceptible to shrinkage, and are not likely to swell.

The risks and design and construction considerations associated with managing shallow silt and the swell and shrinkage risks at this site are considered typical and normal with grade supported slabs in Winnipeg. Furthermore, excavation of the crawlspace is likely to result in full depth removal of the silt over a large portion of the footprint.

If soft clay/silt conditions are encountered and drying of the subgrade is impractical or does not adequately improve subgrade stability, it is anticipated a stable subgrade for construction could be established via sub-excavation and placement of a 400 mm thick gravel bridging layer underlain by a geotextile separator.

Assuming proper subgrade preparation in accordance with the recommendations outlined in Section 5.5.2 and proper final site grading and surface drainage to mitigate the ingress of runoff into the subgrade, the overall risk of slab movements related to the shrinkage and swell of clay soils can be considered normal, with potential movements estimated in the range of 25 mm to 50 mm under normal site operating and drainage conditions. If the risk for total and differential slab movement is intolerable; either under normal site drainage and operating conditions or an extreme event; then a structurally supported slab should be used.

### 5.8.2 Subgrade Preparation

Subject to the above discussion, recommendations for subgrade preparation for grade supported slabs are as follows:

1. Excavate to design subgrade elevation, which should be taken as the top of slab minus the slab and the recommended minimum gravel structure outlined in Section 5.8.3 or 5.8.4. Further excavation should be conducted as required to remove organic or otherwise unsuitable soils.
2. Stripping and excavation to subgrade design elevation should be completed in such a manner as to minimize disturbance of the subgrade. In this regard, AMEC recommends that excavation be completed using a backhoe equipped with a smooth bladed bucket operating from the edge of the excavation. Further, no construction equipment should be allowed on the exposed subgrade until an assessment of the subgrade has been completed by knowledgeable and experienced geotechnical personnel.
3. All excavated soils should be closely monitored by the geotechnical engineer. All suitable soils should be stockpiled separately for re-use in restoring grades. All materials containing pieces of old pavement and/or organics should be hauled off site for disposal.
4. Once the final subgrade elevation has been achieved, an assessment of the subgrade shall be completed in order to identify any localized loose, ‘weak’, or soft areas prior to trafficking the subgrade and/or prior to fill operations. Ground conditions permitting, assessment of the subgrade should consist of proof-rolling the subgrade with multiple passes of a fully loaded tandem. Notwithstanding, the ability of a subgrade to support proof-roll loads is subject to change throughout construction as a result of changing moisture conditions, and in this regard, proof-rolling may not be possible. The exposed subgrade and feasibility of proof-rolling of the subgrade should be visually evaluated by qualified geotechnical personnel throughout stripping and subgrade preparation operations.
5. Loose, ‘weak’, or soft areas identified either visually or by proof-rolling should be sub-excavated below design subgrade as required to achieve a competent subgrade stratum up to a maximum of 400 mm below grade, and replaced with engineered fill material, as directed by the engineer at the time of construction. Where silt remains at the subgrade elevation, specific backfilling methods and procedures and use of select fill materials (such as 100 mm down crushed limestone) may be required.
6. Protect the exposed subgrade from frost, desiccation (drying), and inundation (wetting) both during and following construction. To reduce accumulation of surface runoff and softening of the subgrade, rough grades should be designed to reduce the potential for ponding of water on the surface and to provide positive drainage towards the perimeter of the subgrade area and/or collection areas as quickly as possible, both during and following subgrade preparation.
7. Depending on disturbance and protection of the subgrade, exposed subgrades that are highly disturbed (i.e. rutted), or desiccated or inundated outside of the acceptable

range of the optimum moisture content (i.e. more than 3 percent below or 5 percent above OMC), should be re-conditioned and re-compacted prior to fill placement. If required, re-conditioning of the subgrade should consist of scarifying the subgrade to a minimum of 200 mm below grade, moisture conditioning dried or wetted soil to between OMC and 5 percent above OMC, and compacted to a minimum of 95 percent of standard Proctor maximum dry density (SPMDD). If excavation to subgrade minimizes disturbance of the subgrade and the subgrade is stable and within an acceptable moisture state, then scarification and re-compaction of the subgrade is not required.

8. Below slab granular fill should be based on actual building loadings and slab performance requirements; however, on a preliminary basis, should consist of a minimum of 150 mm of granular subbase topped by 150 mm of granular base course. The granular materials should be uniformly compacted to a minimum of 100% of SPMDD.
9. Fill materials, if required between the subgrade elevation and the underside of the gravel structures for slabs described in Sections 5.8.3 or 5.8.4 should consist of additional granular subbase. The fill material should be placed in 150 mm thick lifts and uniformly compacted to 98% of SPMDD. Alternatively, suitable approved clay fill could be used, provided that it is free of deleterious materials, moisture conditioned to wet of optimum (preferably two to five percent wet of optimum) and compacted uniformly to 95 percent SPMDD.
10. Place a polyethylene vapour barrier directly below the floor slab.
11. Protect the subgrade from frost, desiccation and inundation prior to, during and after construction.

### 5.8.3 Interior Floor Slabs

Interior grade-supported concrete slabs should be underlain by a minimum gravel structure thickness of 300 mm consisting of 150 mm of base course underlain by 150 mm of granular subbase. Each of the gravel layers should be compacted to 100 percent of SPMDD at ±3 percent of optimum moisture content. It is recommended that the gravel should meet the gradation requirements outlined in City of Winnipeg Specification CW3110. Other gradations may be suitable but should be reviewed by the geotechnical engineer prior to use.

The clay subgrade is estimated to have a subgrade resilient modulus of about 20 MPa, as correlated to a typical California Bearing Ratio (CBR) of 2 percent under soaked conditions. For the purposes of determining concrete slab thicknesses, grade-supported concrete slabs designed on an approved subgrade prepared as outlined in Section 5.8.2 and the gravel base structure outlined above may be designed assuming a subgrade reaction modulus ($k$) of 35 MPa/m.

Further to the above, interior floor slabs should be provided with joints or saw cuts at regular intervals to control and reduce random cracking. All partition walls or equipment founded on the slabs should have a minimum 50 mm thick void space at the top to mitigate damage if the slabs should heave. Interior floor slabs should be free floating, and should be structurally separated from the foundation walls, columns, and foundation walls, except possibly at doorways.

### 5.8.4 Grade-Supported Sidewalks and Apron Slabs

With respect to sidewalks, the upper soils at the site are considered moderately to highly frost susceptible given access to free water. Furthermore, the local clay soils are moderately to highly susceptible to swelling and shrinkage with associated increase or reduction in soil moisture. As such, it is important that adequate site drainage be provided adjacent to exterior sidewalks and aprons to help reduce the accumulation of infiltration water beneath the slabs that could result in frost heave or soil expansion.

Exterior grade-supported concrete slabs should be underlain by a minimum gravel structure thickness of 300 mm consisting of 150 mm of base course underlain by 150 mm of granular subbase. Each of the gravel layers should be compacted to 100 percent of SPMDD at ±3 percent of optimum moisture content. It is recommended that the gravel should meet the gradation requirements outlined in City of Winnipeg Specification CW3110. Other gradations may be suitable but should be reviewed by the geotechnical engineer prior to use.

The clay subgrade is estimated to have a subgrade resilient modulus of about 20 MPa, as correlated to a typical California Bearing Ratio (CBR) of 2 percent under soaked conditions. For the purposes of determining concrete slab thicknesses, grade-supported concrete slabs designed on an approved subgrade prepared as outlined in 5.8.2 and the gravel base structure outlined above may be designed assuming a subgrade reaction modulus (*k*) of 35 MPa/m.

Due to the potential for frost heaving of exterior slabs, all sidewalks and apron slabs should be structurally separate from the structure, and should not be dowelled into the grade beam or the interior slabs except at doorway locations.

Where it is proposed to dowel exterior slabs into structure components, or where frost related movement of the slab is undesirable, rigid insulation could be placed on the subgrade to reduce the depth of frost penetration beneath the slab. In this case, the placement of vertical insulation along the sides of grade beams should be avoided in order to allow heat loss from the building and lessen frost effects. AMEC can provide recommended insulation details for specific exterior grade-supported slabs and apron slabs once the design configurations have been established.

## 5.9 Structurally Supported Floor Slabs & Crawlspaces

Structurally supported floor slabs supported on piles may be supported on piles designed using the recommendations outlined in this report. Subgrade preparation for structurally supported slabs should include the removal of all surface organics. A 150 mm thick, biodegradable cardboard void form should be used below the structural slab to provide the required void space.

Where a crawl space is utilized below a structural floor, the base of the underlying crawl space should be covered with a vapour barrier and a 100 mm thick protective sand cover on top of the vapour barrier. The crawl space should also be heated, ventilated and drained. Depending on the extent of shallow silt encountered within the depth of the crawlspace, combined with requirements to mitigate potential mould within the crawlspace, full depth removal of shallow silt within the crawlspace should be considered as recommended to mitigate preferential seepage and the ingress of moisture into the crawlspace.

## 5.10 Final Site Grading, Surface Drainage, and Subdrainage

Sufficient gradients should be provided to promote surface drainage away from the proposed building in order to reduce the potential for moisture percolation to the foundation elements. Site grading should provide positive drainage away from structures at a minimum gradient of 4 percent for landscaped areas within 3 m of the perimeter of the building; and at a minimum gradient of 2 percent for all pavement areas as well as landscape areas outside of 3 m of the building perimeter. Further to surface grades, all downspouts from the roof of the structure should be discharged away from the building and proper measures (i.e. splashguards) should be provided where necessary to limit the potential for erosion and ponding water at the perimeter of the structure.

Excavations at the perimeter of the structure (grade beams, footings, etc.) should be backfilled with moderately to well compacted fill, and topped with a clay cap a minimum of 0.3 m thick to reduce the potential for surface water infiltration into the slab subgrade or backfill against grade beams. As a recommended minimum, the clay cap in landscape areas along the perimeter of the foundation should extend a minimum of 3.0 m from the foundation perimeter. Where pavement and/or concrete slabs meet the structure, these should be sealed against abutting structural components with a flexible seal, such as an asphaltic bead, to minimize surface water infiltration into the granular layer below the floor slab.

Where a structurally supported main floor slab is provided over a crawl space, a subdrainage system is recommended, particularly if the shallow silt layer is encountered within the depth of the crawlspace. The subdrainage system should consist of a perimeter drain along the exterior perimeter of the building to limit potential groundwater accumulation along foundation elements as well as to limit the ingress of percolating run-off into the underlying crawlspace. Furthermore, an interior drainage collection system consisting of a minimum of one central collection line should be installed within the crawlspace to collect potential seepage into the crawlspace from shallow silt common in Winnipeg clays. The interior drainage collection system should be independent of the perimeter drain system. In order to facilitate gravity drainage of seepage into the crawlspace, grades within the crawlspace should be sloped towards collection lines at a minimum of 2 percent, and ideally, 4 percent.

Perimeter drains and interior collection lines should consist of a minimum 100 mm diameter filter-wrapped perforated PVC pipe placed in trenches backfilled with free draining 40 mm minus drainage gravel. The trenches should be of sufficient width and depth such that a minimum 150 mm thick layer of drainage gravel is maintained above and along the sides of the drain pipe below the finished surface of the crawlspace. Drainage gravel used to backfill the trenches should consist of natural gravel or crushed stone having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic or other deleterious matter, and should meet the following gradation requirement:

**Table 5-5: Drainage Material Grading Requirement**

| Sieve size (Square Openings) | Percent Passing by Weight |
|---|---|
| 40 mm | 100 |
| 25 mm | 50 - 80 |
| 20 mm | 5 - 20 |
| 12.5 mm | 0 - 5 |
| 0.08 mm | 0 – 3 |

Drainage from subdrainage lines should be directed to one or more positive outlets such as a central collecting sump(s); or by gravity flow directly into the sewer system assuming applicable authorities permit. Where drainage is directed to a sump located below the building footprint, interior lateral drainage lines passing beneath the building should consist of solid pipe. Depending on final elevations and site configuration, grading of the crawlspace may necessitate installation of numerous interior drain lines and/or drainage outlets (i.e. sumps) to control slope lengths and drainage line lengths.

## 5.11 Foundation Concrete

Where concrete elements outlined in this report and all other concrete in contact with the local soil will be subjected in service to weathering, sulphate attack, a corrosive environment, or saturated conditions, the concrete should be designed, specified, and constructed in accordance with concrete exposure classifications outlined in the latest edition of CSA standard A23.1, Concrete Materials and Methods of Concrete Construction. In addition, all concrete must be supplied in accordance with current Manitoba and National Building Code requirements.

Based on significant data gathered through previous work in the Winnipeg area, water soluble sulphate concentrations in the soil are typically in the range of 0.2% to 2.0%. As such, the degree of sulphate exposure at the site may be considered as ‘severe’ in accordance with current CSA standards, and the use of sulphate resistance cement (Type HS or HSb) is recommended for concrete in contact with the local soil. Furthermore, air entrainment should be incorporated into any concrete elements that are exposed to freeze-thaw to enhance its durability.

It should be recognized that there may be structural and other considerations, which may necessitate additional requirements for subsurface concrete mix design.

## 5.12 Construction Monitoring and Testing

All engineering design recommendations presented in this report are based on the assumption that an adequate level of testing and monitoring will be provided during construction and that all construction will be carried out by a suitably qualified contractor experienced in foundation and earthworks construction. An adequate level of testing and monitoring is considered to be:

- for earthworks: full-time monitoring and compaction testing.
- for deep foundations: design review and full time monitoring during construction.

- for concrete construction: testing of plastic and hardened concrete in accordance with the latest editions of CSA A23.1 and A23.2; and review of concrete supplier's mix designs for conformance with prescribed and/or performance concrete specifications.

AMEC requests the opportunity to review the design drawings, and the installation of the foundations, to confirm that the geotechnical recommendations have been correctly interpreted. AMEC would be pleased to provide any further information that may be needed during design and to advise on the geotechnical aspects of specifications for inclusion in contract documents.

## 6.0 CLOSURE

The findings and recommendations presented in this report were based on geotechnical evaluation of the subsurface conditions observed during the site investigation described in this report. If conditions other than those reported in this report are noted during subsequent phases of the project, or if the assumptions stated herein are not in keeping with the design, this office should be notified immediately in order that the recommendations can be verified and revised as required. Recommendations presented herein may not be valid if an adequate level of inspection is not provided during construction, or if relevant building code requirements are not met.

The site investigation conducted and described in this report was for the sole purpose of identifying geotechnical conditions at the project Site. Although no environmental issues were identified during the fieldwork, this does not indicate that no such issues exist. If the owner or other parties have any concern regarding the presence of environmental issues, then an appropriate level environmental assessment should be conducted.

Soil conditions, by their nature, can be highly variable across a site. The placement of fill and prior construction activities on a site can contribute to the variability especially in near surface soil conditions. A contingency should always be included in any construction budget to allow for the possibility of variation in soil conditions, which may result in modification of the design and construction procedures.

This report has been prepared for the exclusive use of The City of Winnipeg, and their agents, for specific application to the project described in this report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review and written advice from AMEC. Any use that a third party makes of this report, or any reliance or decisions made based on this report, are the responsibility of those parties. AMEC accepts no responsibility for damages suffered by a third party as a result of decisions made or actions based on this report.

amec foster wheeler

This report has been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty, either expressed or implied, is made.

Respectfully submitted,
**Amec Foster Wheeler Environment & Infrastructure,**
**A Division of Amec Foster Wheeler Americas Limited**

PROVINCE OF MANITOBA
6-APR-2015
K. W. JOHNSON
Member
24333
REGISTERED PROFESSIONAL ENGINEER

APEGM
Certificate of Authorization
Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited
No. 6017 Date: 6 APRIL 2015

Kelly Johnson

Reviewed by:

Kelly Johnson, P. Eng.
Senior Geotechnical Engineer

Harley Pankratz, P.Eng.
Vice President; Eastern Prairies/Northern Alberta

# FIGURES


P:\JOBS\17500'S\17590'S\17599 - GEO. INV. SEVEN OAKS POOL\DRAWINGS\WX17599.DWG

| | | | | |
|---|---|---|---|---|
| CLIENT LOGO | CLIENT<br>CITY OF WINNIPEG | DWN BY: MD<br>CHK'D BY: KJ<br>DATUM: NAD83<br>PROJECTION: UTM Zone 14 U<br>SCALE: AS SHOWN | GEOTECHNICAL INVESTIGATION<br>PROPOSED ADDITION TO SEVEN OAKS POOL<br>ADSUM DRIVE AND JEFFERSON AVENUE<br>WINNIPEG, MANITOBA<br><br>SITE AND TEST HOLE LOCATION PLAN | REV. NO.: A<br>DATE: APRIL 2015<br>PROJECT NO: WX17599<br>FIGURE No. FIGURE 1 |


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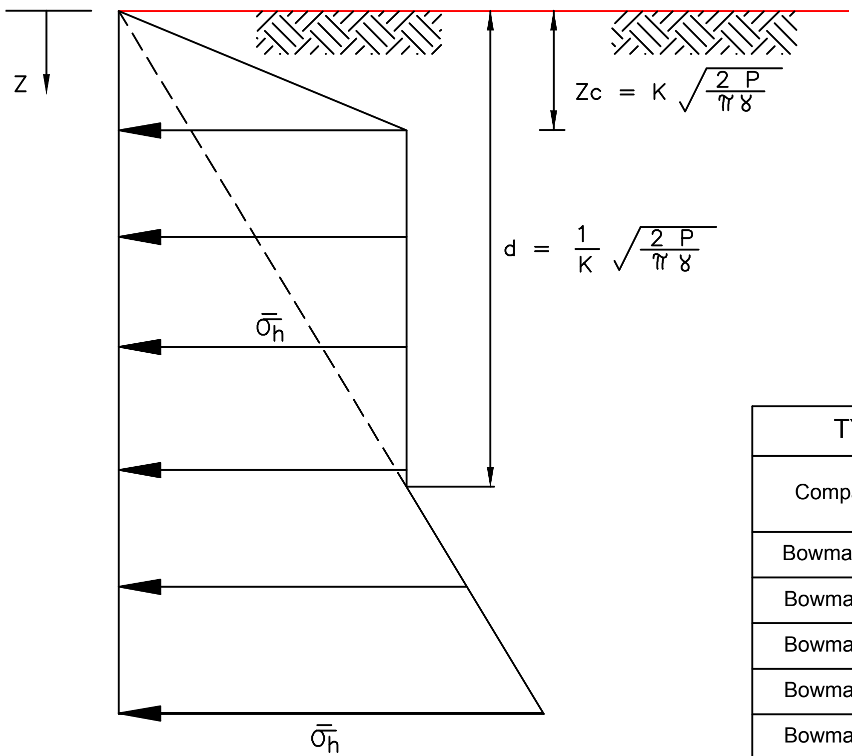



EARTH PRESSURE DISTRIBUTION

FOR $Z_c \leq Z \leq d$

$$\bar{\sigma}_h = \sqrt{\frac{2P\gamma}{\pi}}$$

FOR $Z > d$

$$\bar{\sigma}_h = K \cdot \gamma \cdot Z$$

| TYPICAL COMPACTOR LOADS (P) | | | |
|---|---|---|---|
| Compactor | LOAD (P) kN/m | Compactor | LOAD (P) kN/m |
| Bowmag TSE | 31 | Bowmag BW122PD | 36 |
| Bowmag 60S | 32 | Bowmag 142PDB | 47 |
| Bowmag 65S | 22 | Bowmag 172PDB | 93 |
| Bowmag 75S | 33 | Dynapac LR100 | 42 |
| Bowmag 90S | 39 | Dynapac 2100V | 93 |
| Bowmag 75AD | 20 | Dynapac CA121D | 53 |
| Bowmag 100AD | 20 | Dynapac CA121PD | 54 |
| Bowmag 120AD | 34 | Dynapac CA151 | 80 |
| Bowmag 130AD | 36 | Dynapac CA151D | 80 |
| Bowmag BW122D | 30 | Dynapac CA151PD | 96 |

$$P \text{ (ROLLER LOAD)} = \frac{\text{DEAD WT. OF ROLLER + CENTRIFUGAL FORCE}}{\text{WIDTH OF ROLLER}}$$

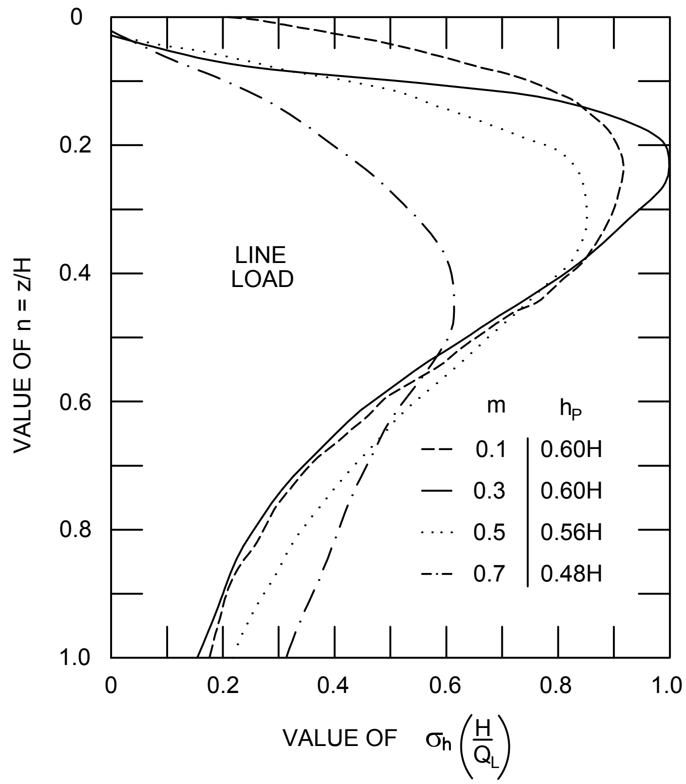
TYPICAL VALUES GIVEN IN TABLE

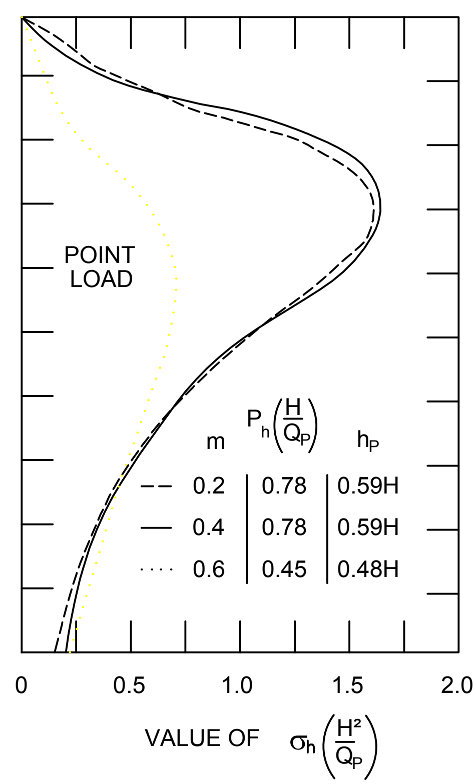
EARTH PRESSURE COEFFICIENTS

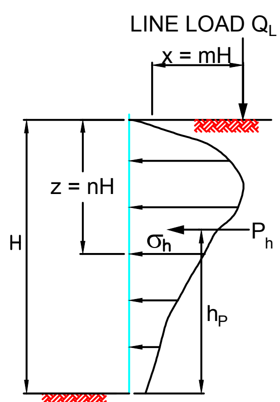
$K = K_o$ ("AT REST") OR $K_a$ (ACTIVE CASE)
(SEE TEXT OF REPORT)

$\gamma$ = SOIL UNIT WEIGHT
(SEE TEXT OF REPORT)

| | | | |
|---|---|---|---|
| CITY OF WINNIPEG | DWN BY: MD | GEOTECHNICAL INVESTIGATION<br>PROPOSED ADDITION TO SEVEN OAKS POOL<br>ADSUM DRIVE AND JEFFERSON AVENUE<br>WINNIPEG, MANITOBA | DATE: APRIL 2015 |
| | CHK'D BY: KJ | | PROJECT No.: WX17599 |
| Amec Foster Wheeler<br>Environment & Infrastructure<br>440 DOVERCOURT DRIVE<br>WINNIPEG, MANITOBA R3Y 1N4<br>PHONE: 204.488.2997 FAX:204.489.8261 | DATUM: - | | REV. No.: A |
| | PROJECTION: - | LATERAL EARTH PRESSURES INDUCED BY COMPACTION | FIGURE No.: FIGURE 2 |
| | SCALE: NOT TO SCALE | | |




FOR $m \le 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16 + n^2)^2}$$

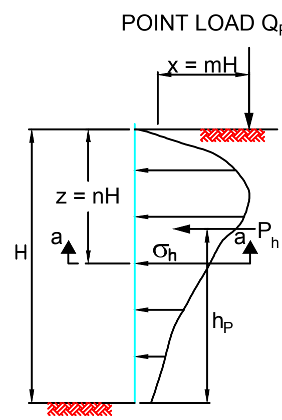
$$P_h = 0.55\ Q_L$$

FOR $m > 0.4$:

$$\sigma_h \left(\frac{H}{Q_L}\right) = \frac{1.28m^2n}{(m^2 + n^2)^2}$$

$$\text{RESULTANT } P_h = \frac{0.64\ Q_L}{(m^2 + 1)}$$

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



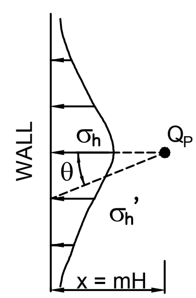

FOR $m \le 0.4$:

$$\sigma_h \left(\frac{H^2}{Q_P}\right) = \frac{0.28n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$:

$$\sigma_h \left(\frac{H^2}{Q_P}\right) = \frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma_h' = \sigma_h \ \cos^2 (1.1\theta)$$




SECTION a-a
PRESSURES FROM POINT LOAD
(BOUSSINESQ EQUATION
MODIFIED BY EXPERIMENT)

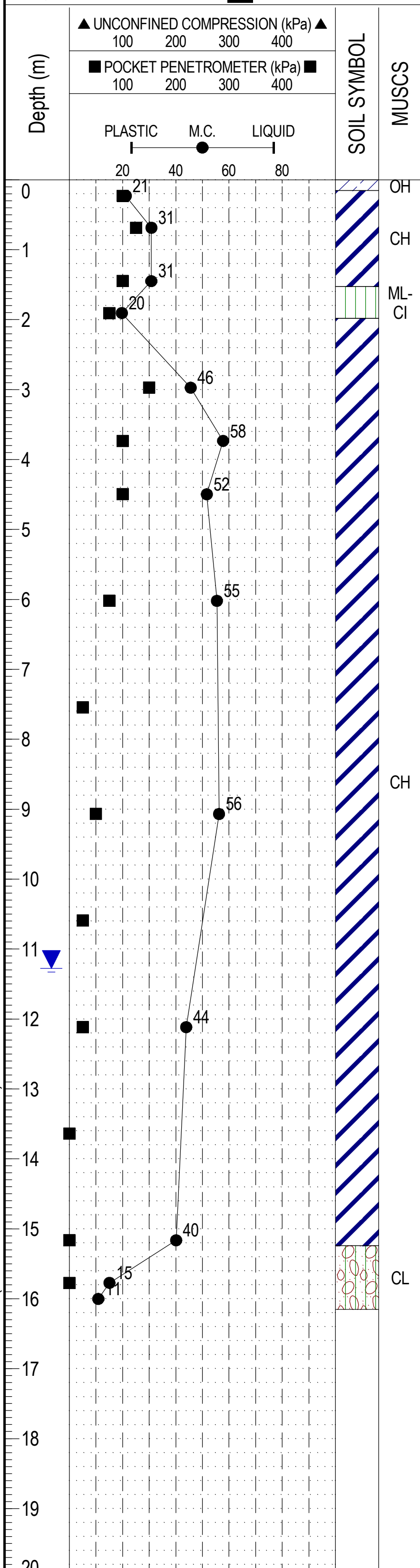
| | | | |
|---|---|---|---|
| CITY OF WINNIPEG | DWN BY: MD | GEOTECHNICAL INVESTIGATION<br>PROPOSED ADDITION TO SEVEN OAKS POOL<br>ADSUM DRIVE AND JEFFERSON AVENUE<br>WINNIPEG, MANITOBA | DATE: APRIL 2015 |
| | CHK'D BY: KJ | | PROJECT No.: WX17599 |
| Amec Foster Wheeler<br>Environment & Infrastructure<br>440 DOVERCOURT DRIVE<br>WINNIPEG, MANITOBA R3Y 1N4<br>PHONE: 204.488.2997 FAX:204.489.8261 | DATUM: - | | REV. No.: A |
| | PROJECTION: - | LATERAL PRESSURES DUE TO SURCHARGE<br>POINT AND LINE LOADS | FIGURE No.: FIGURE 3 |
| | SCALE: NOT TO SCALE | | |

# APPENDIX A

| | | |
|---|---|---|
| PROJECT: Geotechnical Investigation - Seven Oaks Pool | DRILLED BY: Maple Leaf Drilling | BORE HOLE NO: **TH01** |
| CLIENT: CIty of Winnipeg | DRILL TYPE: DR-150 | PROJECT NO: WX17599 |
| LOCATION: 444 Adsum Drive | DRILL METHOD: 125mm SSA | ELEVATION: 232.37 m |

SAMPLE TYPE: Shelby Tube, No Recovery, SPT (N), Grab Sample, Split-Pen, Core

BACKFILL TYPE: Bentonite, Pea Gravel, Drill Cuttings, Grout, Slough, Sand




| MUSCS | SOIL DESCRIPTION | SAMPLE NO | SPT (N) | COMMENTS |
|---|---|---|---|---|
| OH | Topsoil, organics 150mm, grass covered | 1 | | |
| CH | CLAY FILL, silty, high plasticity, damp, stiff, brown, frequent silt inclusions, frozen to 0.9m | 2, 3 | | |
| ML-CI | SILT, some clay to clayey, low to medium plastic, moist, firm, tan | | | |
| CH | CLAY, silty, high plasticity, moist, stiff, brown,<br>- below 3.6m, very moist<br>- below 5.5m, firm, grey, frequent silt inclusions<br>- below 10.7m, trace gravel, moist to wet, soft<br>- below 12.2m, very soft<br>- below 14.6m, trace sand | 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14 | | |
| CL | CLAY TILL, CLAY, silty, gravelly, some sand, low plasticity, wet, soft, grey | 15, 16 | 90/275 | Spoon refusal at 16.2m |

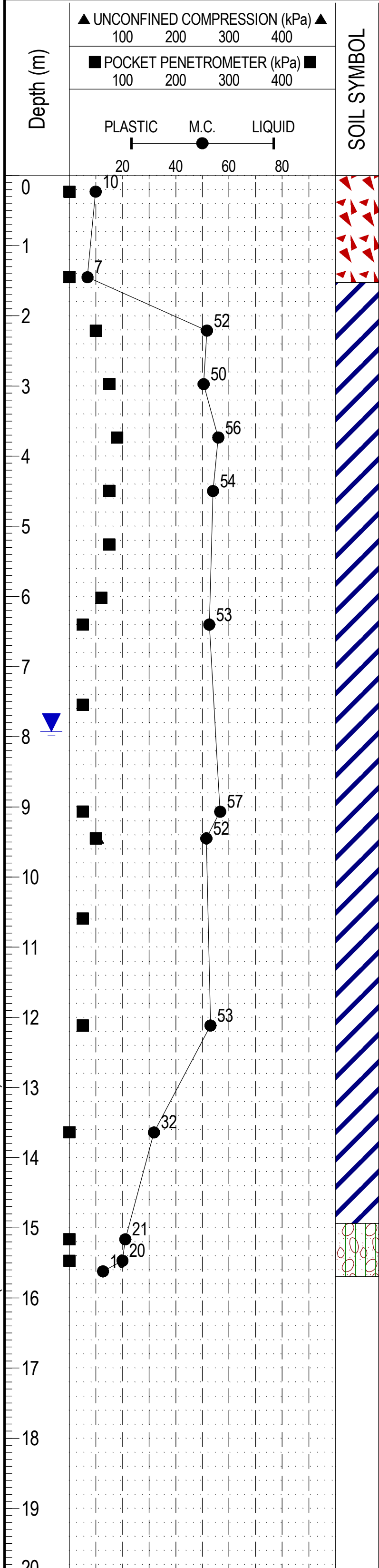
AUGER & SPT REFUSAL at 16.2m BELOW GRADE
- Seepage observed from 14.6m to 16.2m below grade.
- Test hole open to 15.5m below grade upon completion of drilling.
- Water level measured at 11.3 m below grade approximately 10 minutes following completion of drilling.
- Test hole backfilled with auger cuttings and bentonite.

ELEVATION (m): 232, 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213

| | | |
|---|---|---|
| **AMEC Foster Wheeler Environment & Infrastructure**<br>**Winnipeg, Manitoba** | LOGGED BY: BK | COMPLETION DEPTH: 16.2 m |
| | REVIEWED BY: KJ | COMPLETION DATE: 25 February 2015 |
| | Figure No. | Page 1 of 1 |

| | | |
|---|---|---|
| PROJECT: Geotechnical Investigation - Seven Oaks Pool | DRILLED BY: Maple Leaf Drilling | BORE HOLE NO: **TH02** |
| CLIENT: CIty of Winnipeg | DRILL TYPE: DR-150 | PROJECT NO: WX17599 |
| LOCATION: 444 Adsum Drive | DRILL METHOD: 125mm SSA | ELEVATION: 231.62 m |

| | | | | | | |
|---|---|---|---|---|---|---|
| SAMPLE TYPE | Shelby Tube | No Recovery | SPT (N) | Grab Sample | Split-Pen | Core |
| BACKFILL TYPE | Bentonite | Pea Gravel | Drill Cuttings | Grout | Slough | Sand |




| MUSCS | SOIL DESCRIPTION | SAMPLE NO | SPT (N) | COMMENTS |
|---|---|---|---|---|
| GP | GRANULAR FILL, SAND, some gravel, well graded, medium grained, medium density, dry, white to grey, frozen to 1.2m | 1, 2 | | |
| CH | CLAY, silty, high plasticity, moist, brown, firm, occasional silt inclusions<br>- below 2.1m, very moist<br>- below 6.1m soft, grey<br>- below 12.8m, trace to some gravel, wet, very soft | 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15 | | **UCS Test Results (S12):**<br>Compressive Strength: 54 kPa<br>Strain at Failure, $e_{100}$: 3.8%<br>Strain at 50 % UCS, $e_{50}$: 0.9%<br>Bulk Density: 1773 kg/m$^3$<br>Dry Density: 1171 kg/m$^3$ |
| CL | CLAY TILL, CLAY AND SILT, some gravel, trace sand, low plasticity, wet, very soft | 16, 17, 18 | 53/140 | Spoon refusal at 15.7m |

AUGER & SPT REFUSAL at 15.7m BELOW GRADE
- Seepage observed below 15.2m below grade during drilling.
- Sloughing observed below 12.8m below grade during drilling.
- Test hole open to 7.9m below grade upon completion of drilling.
- Water level measured at 7.9 m below grade approximately 10 minutes following completion of drilling.
- Test hole backfilled with auger cuttings and bentonite.

ELEVATION (m): 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212

| | | |
|---|---|---|
| **AMEC Foster Wheeler Environment & Infrastructure**<br>**Winnipeg, Manitoba** | LOGGED BY: BK | COMPLETION DEPTH: 15.7 m |
| | REVIEWED BY: KJ | COMPLETION DATE: 25 February 2015 |
| | Figure No. | Page 1 of 1 |

| | | |
|---|---|---|
| PROJECT: Geotechnical Investigation - Seven Oaks Pool | DRILLED BY: Maple Leaf Drilling | BORE HOLE NO: **TH03** |
| CLIENT: CIty of Winnipeg | DRILL TYPE: DR-150 | PROJECT NO: WX17599 |
| LOCATION: 444 Adsum Drive | DRILL METHOD: 125mm SSA | ELEVATION: 232.01 m |

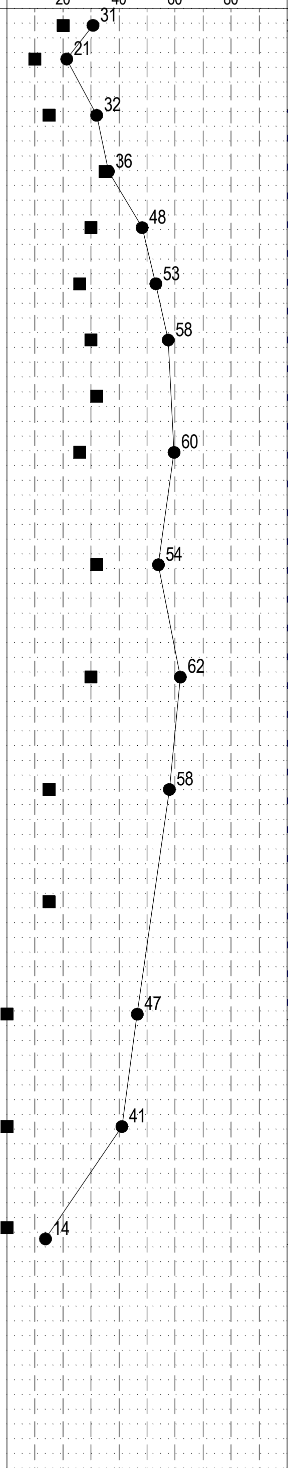
SAMPLE TYPE: Shelby Tube, No Recovery, SPT (N), Grab Sample, Split-Pen, Core

BACKFILL TYPE: Bentonite, Pea Gravel, Drill Cuttings, Grout, Slough, Sand

▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400

■ POCKET PENETROMETER (kPa) ■ 100 200 300 400

PLASTIC M.C. LIQUID 20 40 60 80




| Depth (m) | MUSCS | SOIL DESCRIPTION | SAMPLE NO | SPT (N) | COMMENTS | ELEVATION (m) |
|---|---|---|---|---|---|---|
| 0 | OH | GRANULAR FILL, SAND, some gravel, well graded, medium grained, medium density, dry, white to grey, frozen | 1 | | | |
| | CH | CLAY FILL, silty, trace sand, high plasticity, dry, firm, black, frozen | 2 | | | |
| 1 | ML | SILT, some clay to clayey, low to medium plasticity, moist, firm, tan | 3 | | | 231 |
| 2 | | CLAY, silty, high plasticity, moist, stiff, brown, occasional silt inclusions | 4 | | | 230 |
| 3 | | | 5 | | | 229 |
| 4 | | - below 3.6m, very moist | 6 | | | 228 |
| | | | 7 | | | |
| 5 | | | 8 | | | 227 |
| 6 | | - below 5.5m, grey | 9 | | | 226 |
| 7 | | | | | | 225 |
| | CH | | 10 | | | |
| 8 | | | | | | 224 |
| 9 | | | 11 | | | 223 |
| 10 | | | | | | 222 |
| | | - below 10.4m, firm | 12 | | | |
| 11 | | | | | | 221 |
| 12 | | | 13 | | | 220 |
| 13 | | | | | | 219 |
| | | | 14 | | | |
| 14 | | CLAY TILL, CLAY and SILT, some gravel, trace sand, low plasticity, wet, soft, grey | | | | 218 |
| 15 | CH | | 15 | | | 217 |
| 16 | | | 16 | | | 216 |
| | | | 17 | 52/125 | Spoon refusal at 16.8m | |
| 17 | | AUGER & SPT REFUSAL at 16.8m BELOW GRADE | | | | 215 |
| 18 | | | | | | 214 |
| 19 | | | | | | 213 |
| 20 | | | | | | |

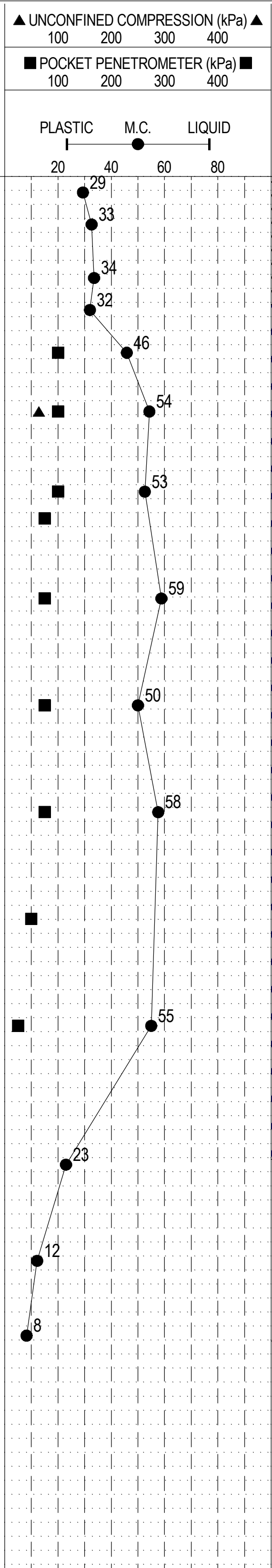
AUGER & SPT REFUSAL at 16.8m BELOW GRADE
- Seepage observed below 15.2m below grade during drilling.
- Sloughing observed below 13.7m below grade during drilling.
- Test hole open to 14.6m below grade upon completion of drilling.
- Water level measured at 11.0 m below grade approximately 10 minutes following completion of drilling.
- Test hole backfilled with auger cuttings and bentonite.

**AMEC Foster Wheeler Environment & Infrastructure**
**Winnipeg, Manitoba**

| | |
|---|---|
| LOGGED BY: BK | COMPLETION DEPTH: 16.8 m |
| REVIEWED BY: KJ | COMPLETION DATE: 25 February 2015 |
| Figure No. | Page 1 of 1 |

| PROJECT: Geotechnical Investigation - Seven Oaks Pool | DRILLED BY: Maple Leaf Drilling | BORE HOLE NO: **TH04** |
|---|---|---|
| CLIENT: CIty of Winnipeg | DRILL TYPE: Renegade | PROJECT NO: WX17599 |
| LOCATION: 444 Adsum Drive | DRILL METHOD: 125mm SSA | ELEVATION: 232.15 m |

| | | | | | | |
|---|---|---|---|---|---|---|
| SAMPLE TYPE | Shelby Tube | No Recovery | SPT (N) | Grab Sample | Split-Pen | Core |
| BACKFILL TYPE | Bentonite | Pea Gravel | Drill Cuttings | Grout | Slough | Sand |




| Depth (m) | MUSCS | SOIL DESCRIPTION | SAMPLE NO | SPT (N) | COMMENTS |
|---|---|---|---|---|---|
| 0 | CH | CLAY FILL, silty, some sand and gravel, high plastic, frozen, grey, some organics to 0.3m, grass covered | 1, 2 | | |
| | | - damp to moist, friable below 1.2m | 3 | | |
| | CH | CLAY, silty, high plastic, moist, firm, dark grey mottled brown, frequent silt inclusions | 4, 5 | | |
| | | - below 2.4m, firm to stiff, grey mottled brown, occasional silt inclusions | | | |
| | | - below 3.0m, very moist | 6 | | **UCS Test Results (S6):** Compressive Strength: 64 kPa; Strain at Failure, $e_{100}$: 10.3%; Strain at 50 % UCS, $e_{50}$: 1.8%; Bulk Density: 1779 kg/m$^3$; Dry Density: 1153 kg/m$^3$ |
| | | - below 4.6m, occasional sulphate inclusions | 7, 8 | | |
| | | - below 5.2m, firm | 9, 10 | | |
| | | - below 9.1m, soft to firm | 11 | | |
| | | - below 10.7m, soft | 12, 13 | | |
| | ML | SILT TILL, clayey, low plastic, very moist, loose (inferred), grey | 14 | | |
| | | - trace clay, some sand, damp below 14.9m | 15 | | |
| | | - compact below 15.2m | 16 | 14 | |
| | | - sand layer at 15.5m | 17 | | |
| 16.6 | | AUGER REFUSAL at 16.6m BELOW GRADE | | | |

- Seepage observed below 15.2m below grade during drilling.
- Sloughing observed below 12.2m below grade during drilling.
- Test hole open to 16.2m below grade upon completion of drilling.
- Water level measured at 9.4 m below grade approximately 10 minutes following completion of drilling.
- Test hole backfilled with auger cuttings and bentonite.

Depth (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

ELEVATION (m): 232 231 230 229 228 227 226 225 224 223 222 221 220 219 218 217 216 215 214 213

| AMEC Foster Wheeler Environment & Infrastructure Winnipeg, Manitoba | LOGGED BY: BK | COMPLETION DEPTH: 16.6 m |
|---|---|---|
| | REVIEWED BY: KJ | COMPLETION DATE: 26 February 2015 |
| | Figure No. | Page 1 of 1 |

# EXPLANATION OF TERMS AND SYMBOLS

The terms and symbols used on the borehole logs to summarize the results of field investigation and subsequent laboratory testing are described in these pages.

It should be noted that materials, boundaries and conditions have been established only at the borehole locations at the time of investigation and are not necessarily representative of subsurface conditions elsewhere across the site.

**TEST DATA**

Data obtained during the field investigation and from laboratory testing are shown at the appropriate depth interval.

Abbreviations, graphic symbols, and relevant test method designations are as follows:

| | | | |
|---|---|---|---|
| *C | Consolidation test | *ST | Swelling test |
| $D_R$ | Relative density | TV | Torvane shear strength |
| *k | Permeability coefficient | VS | Vane shear strength |
| *MA | Mechanical grain size analysis and hydrometer test | w | Natural Moisture Content (ASTM D2216) |
| | | $w_l$ | Liquid limit (ASTM D 423) |
| N | Standard Penetration Test (CSA A119.1-60) | $w_p$ | Plastic Limit (ASTM D 424) |
| $N_d$ | Dynamic cone penetration test | $E_f$ | Unit strain at failure |
| NP | Non plastic soil | $\gamma$ | Unit weight of soil or rock |
| pp | Pocket penetrometer strength | $\gamma_d$ | Dry unit weight of soil or rock |
| *q | Triaxial compression test | $\rho$ | Density of soil or rock |
| $q_u$ | Unconfined compressive strength | $\rho_d$ | Dry Density of soil or rock |
| *SB | Shearbox test | $C_u$ | Undrained shear strength |
| $SO_4$ | Concentration of water-soluble sulphate | → | Seepage |
| | | $\underline{\blacktriangledown}$ | Observed water level |

* The results of these tests are usually reported separately

Soils are classified and described according to their engineering properties and behaviour.

The soil of each stratum is described using the Unified Soil Classification System[1] modified slightly so that an inorganic clay of "medium plasticity" is recognized.

The modifying adjectives used to define the actual or estimated percentage range by weight of minor components are consistent with the Canadian Foundation Engineering Manual[2].

Relative Density and Consistency:

| Cohesionless Soils | |
|---|---|
| Relative Density | SPT (N) Value |
| Very Loose | 0-4 |
| Loose | 4-10 |
| Compact | 10-30 |
| Dense | 30-50 |
| Very Dense | >50 |

| Cohesive Soils | | |
|---|---|---|
| Consistency | Undrained Shear Strength $c_u$ (kPa) | Approximate SPT (N) Value |
| Very Soft | 0-12 | 0-2 |
| Soft | 12-25 | 2-4 |
| Firm | 25-50 | 4-8 |
| Stiff | 50-100 | 8-15 |
| Very Stiff | 100-200 | 15-30 |
| Hard | >200 | >30 |

Standard Penetration Resistance ("N" value)
The number of blows by a 63.6kg hammer dropped 760 mm to drive a 50 mm diameter open sampler attached to "A" drill rods for a distance of 300 mm after an initial penetration of 150 mm.

1 "Unified Soil Classification System", Technical Memorandum 36-357 prepared by Waterways Experiment Station, Vicksburg, Mississippi, Corps of Engineers, U.S. Army. Vol. 1 March 1953.

2 "Canadian Foundation Engineering Manual", 3rd Edition, Canadian Geotechnical Society, 1992.

# MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

| MAJOR DIVISIONS | | | SYMBOLS: USCS | SYMBOLS: GRAPH | SYMBOLS: COLOUR | TYPICAL DESCRIPTION | LABORATORY CLASSIFICATION CRITERIA |
|---|---|---|---|---|---|---|---|
| COARSE GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 75um) | GRAVELS MORE THAN HALF THE COARSE FRACTION LARGER THAN 4.75mm | CLEAN GRAVELS (TRACE OR NO FINES) | GW | | RED | WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES | $C_u=D_{60}/D_{10}>4$; $C_c=(D_{30})^2/(D_{10}xD_{60})$ = 1 to 3 |
| | | | GP | | RED | POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES | NOT MEETING ABOVE REQUIREMENTS |
| | | DIRTY GRAVELS (WITH SOME OR MORE FINES) | GM | | YELLOW | SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES | ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4 |
| | | | GC | | YELLOW | CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES | ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7 |
| | SANDS MORE THAN HALF THE COARSE FRACTION SMALLER THAN 4.75mm | CLEAN SANDS (TRACE OR NO FINES) | SW | | RED | WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES | $C_u=D_{60}/D_{10}>6$; $C_c=(D_{30})^2/(D_{10}xD_{60})$ = 1 to 3 |
| | | | SP | | RED | POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES | NOT MEETING ABOVE REQUIREMENTS |
| | | DIRTY SANDS (WITH SOME OR MORE FINES) | SM | | YELLOW | SILTY SANDS, SAND-SILT MIXTURES | ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4 |
| | | | SC | | YELLOW | CLAYEY SANDS, SAND-CLAY MIXTURES | ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7 |
| FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75um) | SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT | $W_L < 50\%$ | ML | | GREEN | INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY | CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW) |
| | | $W_L > 50\%$ | MH | | BLUE | INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SAND OR SILTY SOILS | |
| | CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT | $W_L < 30\%$ | CL | | GREEN | INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS | |
| | | $30\% < W_L < 50\%$ | CI | | GREEN-BLUE | INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS | |
| | | $W_L > 50\%$ | CH | | BLUE | INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS | |
| | ORGANIC SILTS & CLAYS BELOW "A" LINE | $W_L < 50\%$ | OL | | GREEN | ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY | WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY |
| | | $W_L > 50\%$ | OH | | BLUE | ORGANIC CLAYS OF HIGH PLASTICITY | |
| HIGHLY ORGANIC SOILS | | | PT | | ORANGE | PEAT AND OTHER HIGHLY ORGANIC SOILS | STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE |

## SPECIAL SYMBOLS

| | | | |
|---|---|---|---|
| LIMESTONE | | OILSAND | |
| SANDSTONE | | SHALE | |
| SILTSTONE | | FILL (UNDIFFERENTIATED) | |

## SOIL COMPONENTS

| FRACTION | U.S. STANDARD METRIC SIEVE SIZE: PASSING | U.S. STANDARD METRIC SIEVE SIZE: RETAINED |
|---|---|---|
| GRAVEL | | |
| COARSE | 76mm | 19mm |
| FINE | 19mm | 4.75mm |
| SAND | | |
| COARSE | 4.75mm | 2.00mm |
| MEDIUM | 2.00mm | 425μm |
| FINE | 425μm | 75μm |
| FINES (SILT OR CLAY BASED ON PLASTICITY) | 75μm | |

| DEFINING RANGES OF PERCENT BY WEIGHT OF MINOR COMPONENTS: PERCENT | DESCRIPTOR |
|---|---|
| 35 - 50 | AND |
| 30 - 35 | Y / EY |
| 10 - 20 | SOME |
| 1 - 10 | TRACE |

## OVERSIZED MATERIAL

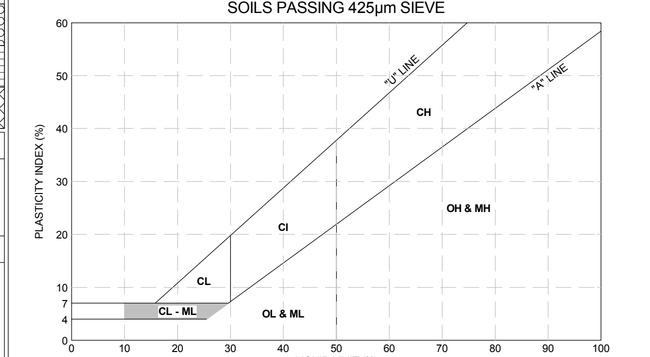
ROUNDED OR SUBROUNDED:

COBBLES 76mm to 200mm
BOULDERS > 200mm

NOT ROUNDED:

ROCK FRAGMENTS ? 76mm
ROCKS > 0.76 CUBIC METRE IN VOLUME

PLASTICITY CHART FOR SOILS PASSING 425μm SIEVE




NOTES:
1. ALL SIEVE SIZES MENTIONED ARE U.S. STANDARD ASTM E.11.
2. COARSE GRAINED SOILS WITH TRACE TO SOME FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH TRACE TO SOME CLAY.
3. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

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