GEOTECHNICAL INVESTIGATION
PROPOSED ADMINISTRATIVE BUILDING
BRADY LANDFILL
WINNIPEG, MANITOBA

Submitted to:

City of Winnipeg
Municipal Accommodations Division
City of Winnipeg

Attention: Mr. Andrew Urbanowicz , CET

Submitted by:

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1.0 INTRODUCTION

As authorized by Mr. Andrew Urbanowicz, CET, representing the City of Winnipeg (COW), Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited (Amec Foster Wheeler), completed a geotechnical investigation for a proposed Administrative Building at the Brady Landfill in Winnipeg, Manitoba.

The scope of work was provided in Amec Foster Wheeler's proposal WPG2016.403 dated 10 August 2016. The purpose of the investigation was to evaluate the soil and groundwater conditions at the site and on this basis, provide geotechnical recommendations for the design and construction of cast-in-place pile foundations, asphalt pavements and floor slabs.

2.0 PROPOSED FACILITIES

Based on limited information provided by COW, details of the proposed Administrative building development are as follows:;

- a 5,000 sq. ft., single story steel framed building
- Paved parking areas surrounding building
- Slab-on-grade main floor

Building loads were not provided.

3.0 SITE CONDITIONS

The site is located directly north of the existing Commercial Vehicle access road to Brady Landfill as noted on Figure 1 attached to this report. At the time of the geotechnical investigation, the site was a vacant grass covered field. Grassed fields were present to the north and east of the proposed development with the commercial access located to the south. The topography of the site was generally flat lying, with drainage assumed to be overland towards the ditch on the west side of the site.

4.0 FIELD INVESTIGATION

Prior to initiating drilling, Amec Foster Wheeler contacted the various public utilities to ensure drilling could be completed without contact with underground services. All field activities were completed without incident.

The test holes were advanced at the site on 07 November 2016. The test holes were advanced to depths that ranged from 3.1m to 14.6 m from grade and were drilled using a track mounted Acker MP5 geotechnical drill rig equipped with 125 mm diameter solid stem augers, operated by Maple Leaf Drilling Ltd. of Winnipeg, Manitoba, under the supervision of Mr. Caolan McEvoy, CET of Amec Foster Wheeler.

All soils observed during drilling of the test hole were visually classified on site according to the Modified Unified Soil Classification System. Groundwater and drilling conditions, as well as pertinent subsurface observations, were also recorded at the time of the investigation.

Sampling consisted of disturbed samples obtained from the auger cuttings and relatively undisturbed Shelby tube samples recovered at select depths and locations in the clay. Additionally, split spoon sample were collected within the glacial till deposit, in conjunction with Standard Penetration Testing (SPT's). The number of blows required to drive the SPT sampler through a depth interval of 300 mm was recorded, and is shown on the test hole logs as the SPT (N) value. Field testing included Pocket Penetrometer tests on auger samples to identify the approximate shear strength profile of the clay soils.

After completion of drilling, the test holes were left open for approximately ten minutes to observe short-term sloughing and seepage conditions. The test holes were then backfilled with auger cuttings and a layer of bentonite, with excess auger cuttings left on the surface adjacent the test hole locations. All soil samples obtained during the field investigation were labelled, sealed to limit moisture loss and transported to Amec Foster Wheeler's Winnipeg office for further visual examination and laboratory testing.

Detailed test hole logs summarizing the sampling, field and laboratory test results, and subsurface conditions encountered are presented as Figures 2 to 6 in Appendix A. Due to the method by which the soil cuttings are returned to the surface, actual depths may vary by ± 0.3 m from those recorded on the test hole log.

5.0 LABORATORY TESTING

Soil samples transported to the laboratory were visually examined by Amec Foster Wheeler's Project Engineer to supplement and confirm the field classifications. Selected soil samples recovered during the field investigation were tested to determine their natural moisture content, Atterberg limits and unconfined compressive strength and the test results are provided on the test hole logs, Figures 2 to 6 in Appendix A.

6.0 SUBSURFACE CONDITIONS

The soil stratigraphy at the site, as noted in descending order from the ground surface at the test hole locations, was as follows:

- Organic Clay
- Clay with Silt Layer
- Glacial Till

<u>Organic Clay</u>

Organic clay was noted at the surface in all test holes. The organic clay, also more commonly referred to as topsoil, was grass covered and consisted of a clay that was silty, contained some sand, high to medium plastic, moist, firm, dark grey to black, with frequent rootlets. The organic clay was generally about 150mm thick.

<u>Clay with Silt Layer</u>

Lacustrine clay was present below the organic clay and extending to a depth of about 12.8m in test hole TH01 and 13.1m in test hole TH02. The clay extended to the termination depth of

3.1m in the remaining test holes. In general the clay was silty, high plastic, moist, stiff and brown becoming grey and firm to soft with increasing depth, with occasional silt lens.

A silt layer was observed immediately below the organic clay in TH03 and TH04, and within the clay in test hole TH01 (between 0.8 m to 1.5 m). The silt ranged in thickness from 150 mm to 750 mm. The silt contained trace amounts of sand and was low plastic, very moist, very soft and tan-brown.

Glacial Till

In test hole TH01 and TH02 only, glacial till was noted below the lacustrine clay beginning at about 12.8m to 13.1m from grade and extending to auger refusal at a depths of about 14.0 to 14.2 m below current site grade. The glacial silt till was sandy, contained some gravel with trace amounts of clay. The silt till was moist, compact (inferred) becoming dense with depth and tan to brown.

A detailed description of the soil profile encountered, and the results of field and laboratory testing, is provided in the test hole logs, Figures 2 to 6, Appendix A.

6.1 SLOUGHING AND SEEPAGE CONDITIONS

The test holes were left open for approximately 10 minutes after completion of drilling to measure short term sloughing and seepage. Table 1 below summarizes the seepage and sloughing conditions noted during and at completion of drilling.

Table 1: Seepage and Sloughing Conditions

Test Hole Number	Depth of Seepage during drilling	Depth of Sloughing during drilling	Depth to slough upon completion	Depth to water upon completion
TH01	14.5 m from silt till layer	14.5 m from silt till layer	14.5 m	None observed
TH02	None observed	13.1m from silt till layer	14.0 m	None observed
TH03	None observed	None observed	3.1 m	None observed
TH04	None observed	None observed	3.1 m	None observed
TH05	None observed	None observed	3.1 m	None observed

It should be noted that only short-term seepage and sloughing conditions were observed and that ground water levels can fluctuate annually, seasonally or as a result of construction activity.

6.2 POWER AUGER REFUSAL

Power auger refusal was encountered in test holes TH01 and TH02 at a depths of about 14.6 m and 14.0 m below grade, respectively, at the time of the field investigation.

7.0 RECOMMENDATIONS

7.1 FOUNDATIONS

Given the soil conditions encountered, drilled cast-in-place concrete friction piles and driven precast concrete piles are both considered suitable foundation options for the proposed building. Shallow foundations (i.e. footings) are not recommended due to typically poor performance caused by consolidation settlement, and shrinkage and swelling of the high plastic clay.

7.2 CAST-IN-PLACE CONCRETE PILES

7.2.1 Axial Compressive Resistance for Single Cast-In-Place Concrete Friction Piles

Bored straight shaft, cast-in-place concrete friction piles, embedded in the native clay deposit to depths of up to 12 m below existing grade, are considered suitable for support of the proposed building. Due to difficulties with ensuring a clean base using typical installation methods, end bearing resistance should be neglected in the design. Furthermore pile depths should be limited to about 12 m given the potential for groundwater inflow at the base of the pile.

The ultimate vertical compressive axial resistance of a single, bored straight shaft cast-in-place concrete pile may be determined using the unit shaft friction values provided in Table 2.

Table 2: Ultimate Axial Compressive Resistance, Cast-in-Place Concrete Piles

Depth Below Existing Grade (m)	Assumed Soil Type	Ultimate Unit Shaft Friction (kPa)
0 to X[1]	All	0
X to 12.0	Clay	37

Note:
1. X = greatest of:
   - 1.5 m below existing grade for interior piles in heated buildings; or
   - 2.5 m below finished grade for perimeter piles, or piles located in unheated areas

To obtain the factored geotechnical resistance at the Ultimate Limit State (ULS) for axial compressive loading conditions, the ultimate unit shaft resistance should be multiplied by a geotechnical resistance factor, Φ, of 0.4.

The ultimate values provided apply only to shaft friction in the clay, and neglect the relatively small contribution from end bearing. Where end bearing resistance in the clay is to be considered, this office should be contacted; however, would likely require the use of alternative installation methods.

7.2.2 Serviceability Limit State for Cast-In-Place Concrete Piles

Provided that appropriate construction practices are followed, an estimated pile displacement of 0.05% to 0.15% of the pile diameter, plus the elastic shortening of the pile due to the compressive load acting on the pile, can be assumed for drilled cast-in-place concrete piles designed according to the parameters defined in Table 2. Under these conditions, the Serviceability Limit State (SLS) pile resistance may be taken to be equal to the factored geotechnical resistance at the ULS. Long term settlements may be greater for large pile groups. Where more stringent settlement magnitudes are required, this office should be contacted for a revised SLS pile resistance.

7.2.3 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 7.5.

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 uplift resistance of CIP concrete piles due to shaft friction can be determined using the ultimate unit shaft friction values outlined in Table 2.

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

7.2.4 Design and Installation of Cast-In-Place Concrete Piles

In the case of conventionally bored straight shaft cast-in-place piles, the following design and construction procedures should be adopted:

- 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 7.5.
- A void space (minimum of 150 mm thick) should be constructed, using a compressible and biodegradable material, below all piles caps and grade beams to accommodate movements of the underlying soil.
- Sloughing and seepage is anticipated for bored concrete piles at some locations given the presence of the shallowsilt layer. Temporary steel casings should be on and hand and used as required in the pile holes 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 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 holes should be managed using a bailing bucket or a submersible pump subject to actual field conditions.
- In accordance with the National Building Code, qualified and experienced geotechnical personnel of the engineer of record should be on site to monitor pile installation on a full time basis, and should maintain complete and accurate records of the pile installations.

7.2.5 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 Foster Wheeler can undertake evaluation of lateral pile capacity if required, upon request.

7.3 DRIVEN PRECAST CONCRETE PILES

7.3.1 Ultimate Limit State

Driven precast concrete piles are also considered well suited for the site conditions. The precast concrete piles should be driven to practical refusal in the dense glacial till, or on the underlying limestone bedrock. Provided that a hammer capable of delivering a minimum energy of 40 kJ per blow is used, single pre-cast concrete piles driven to refusal in the dense silt till or on bedrock may be assigned the ultimate axial compressive capacities provided in Table 3.

Table 3: Ultimate Axial Compressive Capacity, Driven Pre-Cast Concrete Piles

Hexagonal Pile Size (mm)	Ultimate Geotechnical Capacity (kN)	Refusal Criteria (blows/25mm)
300	1350	5
350	1650	8
400	2000	12

Based on the *2010 National Building Code of Canada* (NBCC 2010), a geotechnical resistance factor, Φ = 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.

The geotechnical resistance factor can be increased from 0.4 to 0.5 when pile capacities are confirmed using Pile Dynamic Analyzer (PDA) testing. If PDA testing is intended, then testing should be completed on a minimum of 3 piles or 5% of “each” pile type, whichever is greater, for the proposed structure. It should be recognized that designing on a geotechnical resistance factor of 0.5 will require PDA testing at the time of, or preferably prior, to construction. Amec Foster Wheeler’s experience suggests that a larger hammer than noted above is likely necessary in order to prove out larger pile capacities depending on the specific details of the PDA testing (i.e. including hammer configuration), and pile sizes selected, this is particularly case for the 400 mm pile size. It should also be appreciated that results from PDA may be lower than those noted above, and as such some redesign (i.e. additional piles or revision to termination criteria) may be required where PDA testing is undertaken only at the time of construction. Amec Foster Wheeler can provide further details of the PDA testing upon request.

7.3.2 Tensile (Uplift) Resistance – Driven Precast Concrete Piles

In the case of driven straight shaft precast concrete 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 a driven pre-cast pre-stressed concrete pile due to shaft friction can be determined using the ultimate unit shaft friction values outlined in Table 1; 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.

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

7.3.3 Serviceability Limit State – Driven Pre-cast Concrete Piles

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, and assuming good workmanship, the predicted settlement of a precast concrete 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.

7.3.4 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 Foster Wheeler can undertake evaluation of lateral pile capacity if required, upon request.

7.3.5 Design and Installation – Driven Pre-Cast Concrete Piles

The following additional recommendations are provided for the design and installation of driven precast concrete piles:

- Pre-cast concrete piles typically require pre-bore pilot holes to facilitate pile installation. Pre-boring to a maximum depth of about 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. 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. Alternatively, the annulus of over-sized pre-bores shall be backfilled with an approved compacted fill or grout to restore contact between the pile and the adjacent soil required to provide lateral support. If the annulus of over-sized pre-bores cannot adequately be backfilled with an approved fill, then shaft friction and lateral support shall be neglected over the full depth of the pre-bore.
- 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 7.5.
- Recommendations for uplift resistance calculations are provided in Section 7.3.2.

- 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 groups of three piles or less. Where larger pile groups, or closer spacings are required, the details of 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.
- In accordance with the National Building Code, qualified and experienced geotechnical personnel of the engineer of record should be on site to monitor pile installation on a full time basis, and should maintain complete and accurate records of the pile installations.

7.4 PILE GROUP EFFECTS

Amec Foster Wheeler 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 Foster Wheeler should be notified and a review of possible group interactions effects evaluated.

7.5 FROST DESIGN CONSIDERATIONS

7.5.1 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 Foster Wheeler can provide recommended insulation details for specific development conditions upon request.

7.5.2 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.

7.5.2.1 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.

7.5.2.2 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 piles supporting lightly loaded unheated facilities, the piles should be embedded a minimum of 9 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.

7.6 FOUNDATION CONCRETE

Concrete elements including those in contact with the local soil, or that will be subjected in service to weathering, sulphate attack, a corrosive environment, or saturated conditions, should be designed, specified, and constructed in accordance with the 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.

The degree of sulphate exposure in Winnipeg is commonly classified as severe (S-2 exposure classification). On this basis, all concrete in contact with the native soils should be made with sulphate resistant cement (CSA Type HS). The minimum specified 56-day compressive strength and maximum water to cement ratio of the concrete should comply with Tables 2 and 3 of CSA-A23.1. Concrete exposed to freeze-thaw cycles should be air-entrained for freeze-thaw durability in accordance with Table 4, CSA-A23.1.

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

8.0 CONCRETE FLOOR SLABS

Grade supported floor slabs constructed over swell-susceptible clays, such as those present at this site, are generally subject to long term volumetric changes that result in movements which are typically in the order of 25 to 50 mm, however can be as high as 150 mm or more under extreme circumstances. Construction of buildings and pavements generally tends to change natural evaporation routes, often leading to long-term increases in soil moisture content and consequently swelling, particularly within the upper desiccated zone. Pre-existing moisture conditions, which can be influenced by site drainage and recent climatic events, have the greatest effect on the swell potential. Design, maintenance and post construction climate, however, will also significantly influence the actual performance

The underlying lacustrine clays, at the time of our field investigation, had moisture contents that ranged between about 30% and 50% to a depth of 3 m, with moisture contents at approximately 50% below that depth. Atterberg limit test result indicate plastic limits of about 23% with liquid limit results of near 100%, indicative of a highly plastic clay soil. Given these conditions, and based on previous experience, the overall estimated swelling potential of the clay fill at the site is considered to be above average for the Winnipeg area with estimated slab heave potential in the order of about 50 to 75 mm. Greater movements can occur where poor drainage conditions exist and any heave movements which occur could create additional differential slab movements. The above heave movements are based on moisture content data and Amec Foster Wheeler’ experience with similar sites. Where a more accurate estimate of slab movements is required additional lab testing and engineering should be undertaken.

Soft, weak shallow silt was present at each test hole and can cause significant construction issues if not appropriately managed during construction. In order to avoid these issues, excavations should be limited as much as is practical and in the event that the excavations will extend close to the silt zone, placement of a limestone rock bridging layer will likely be necessary to allow for below slab fill placement and compaction, with sub-excavation of soft soils also possible depending on actual final grades.

In summary, a grade supported slab may be used, however, a tolerance for movements will be necessary; or alternatively, a structurally support floor slab should be used.

8.1 GRADE SUPPORTED FLOOR SLABS

Recommendations for construction of concrete slabs on grade are provided below.

1. Excavate to the design subgrade elevation. As a minimum, the depth of excavation is

expected to be equal to the combined thickness of the design slab and proposed underlying granular fill.

2. Based on the results of the test hole logs, the exposed subgrade is expected to consist of either a clay or silt.
3. The subgrade should be protected from frost, desiccation and inundation prior to, during and after construction.
4. The exposed subgrade should be reviewed by geotechnical staff from this office once design subgrade is achieved. The subgrade should be proofrolled with a heavily loaded tandem and where suitable (i.e. no silt noted at surface), lightly compacted to remove effects of excavation disturbance. Where weak zones are noted during or prior to proofrolling, they are typically repaired by sub-excavating to a firm stratum or to a maximum depth of about 400 mm below design subgrade elevation. The subexcavation is then covered with a geotextile fabric and backfilled with a single 400 mm thick lift of, 100 mm to 150 mm diameter crushed limestone rock.
5. Fill materials required between the approved subgrade and the underside of granular section noted below should ideally consist of additional granular limestone sub-base, uniformly compacted in maximum 150 mm thick lifts to 98% of standard Proctor maximum dry density (SPMDD).
6. The below slab granular structure should consist of 150 mm of crushed 20 mm granular limestone base material underlain by 150 mm of 50 mm granular limestone subbase. All granular materials should be placed in maximum 150 mm thick lifts. Subbase should be uniformly compacted to 98% SPMDD and base should be uniformly compacted to 100% SPMDD. All granular fill should meet latest City of Winnipeg Construction specifications (CW3110) for subbase and base course.
7. A polyethylene vapour barrier may be placed below the floor slab to limit moisture migration through the slab. It should be noted that curing problems and curling of the slab at the edges might be encountered where the concrete slab is cast directly on the poly.

To limit the effects of slab movements on the building structure, the following provisions are recommended:

I. Design equipment and partition walls bearing on the slab with a void space to minimize the potential for structural damage if the slab heaves.
II. Provide control joints at regular intervals in the slab to reduce random cracking.
III. Construct the floor independent of structural elements by the use of isolation joints.

8.2 STRUCTURAL FLOOR SLABS

Where the above noted potential for movements of a grade supported slab is not acceptable to the owner, a structural floor should be used, and should be adequately reinforced and supported on piles. Preparation for construction should ensure the complete removal of any organic soils from the building footprint, to prevent the build-up of methane gas. The structural floor should be underlain by either a 150 mm thick compressible and biodegradable cardboard void form material, or a vented and heated crawl space. If a crawl space is to be considered, this office should be contacted for additional recommendations.

9.0 ASPHALT PAVEMENT

The construction and performance of asphalt and concrete pavements at this site will generally have similar concerns as those presented for grade supported floor slabs in regard to the presence of soft, weak and frost susceptible silt. Where possible, final grades in the parking areas should be as high as practical in order to provide cover over the frost susceptible silt material.

The asphalt pavement sections provided herein are intended as minimums for the design of pavement structures. Design traffic loading information was not available at the time of this report, and in this regard, assumption of anticipated traffic loads and design life were required for providing pavement recommendations. Recommended pavement sections for light duty areas have been provided based on the use of the area by cars with some light truck traffic (i.e. 1-Ton trucks or lighter). Asphalt pavement recommendations for heavy truck traffic areas have also been provided based on these areas being used by fully loaded highway-legal tractor trailers at frequency of less than 5 trucks per day (i.e. delivery and/or garbage trucks). The asphalt pavement sections should be reviewed during detailed design for actual design traffic loading.

Pavement sections have been established based on an assumed effective subgrade resilient modulus (Mr) of 20 MPa, or an approximately equivalent California Bearing Ratio of 2.0 percent. This subgrade resilient modulus is indicative of a relatively low level of subgrade support as is expected during spring thaw when the clay subgrade could exist in a weakened condition. If softened areas are present in the subgrade during construction, excavation and bridging of the soft subgrade may be required as noted in Section 8.1, Point 4.

9.1.1 Subgrade Preparation

For traffic areas and parking lot construction, subgrade preparation should be as follows:

1. Excavate to design subgrade elevation, while further removing any fill or organic soils.
2. The exposed subgrade is expected to consist of native clay or silt. Where clay is noted at the subgrade, proof rolling with a fully loaded tandem truck should be undertaken to detect any soft or weak areas. Any soft or weak areas identified should be replaced or repaired as required prior to placing any granular fill.
3. Generally, the recommendations contained in Section 8.1 (Points 1 to 5) should be followed for pavement subgrade development and gravel placement.

Additional measures recommended to improve long term performance are as follows:

- Maximize drainage slopes.
- Minimize drainage path lengths.

- Provide regular maintenance of the asphalt surface (crack sealing) to prevent water infiltration and subsequent softening of the subgrade.

9.1.2 Asphalt Design Section

Recommended sections for asphalt pavement, constructed on a subgrade prepared as described above, are provided.

Table 4: Asphalt Pavement Design Sections

Material	Standard Duty	Heavy Duty	Compaction Required
Asphalt	65 mm	80 mm	97% of Marshall Density
Base Course (20mm limestone)	100 mm	100 mm	100% of SPMDD
Sub-Base (50 mm limestone)	250 mm	350 mm	98% of SPMDD
Total Thickness	**415 mm**	**530 mm**	**NA**

All granular materials and asphalt concrete should meet City of Winnipeg CW3110 construction specifications for sub-base, base course and asphalt concrete materials.

9.1.3 Concrete Pavement

It is recommended that Portland cement concrete pads be placed at all locations where heavy static wheel loads may exist, such as at loading docks or garbage container pickup areas. At these isolated, unheated locations, frost penetration can be significant and can cause seasonal heave and subsidence.

Concrete pavement areas should be constructed using the recommendations provided in Section 9.1.1 and the following design pavement sections noted in Table 5.

Table 5: Concrete Pavement Design Sections

Material	Standard Duty	Compaction Required
Concrete	200 mm	N/A
Base Course	100 mm	100% of SPMDD
Sub-Base	300 mm	98% of SPMDD
Total Thickness	**600 mm**	**NA**

With respect to maintenance, all pavements, no matter the pavement type, are subject to routine annual maintenance and upkeep essential to maintaining the pavement investment at a specified level of service, and to mitigate the rate of deterioration of the pavement. The annual maintenance is required to repair normal 'wear and tear' and environmental damages, and may include, but not be limited to, crack sealing or seal coating, patching, routing, or dowel joint repairs. In addition, un-maintained pavements are more susceptible to developing serious cracks or structural defects requiring earlier replacement than properly maintained pavements. By mitigating the deterioration of the pavement structure, particularly at depth, a properly maintained pavement could provide for an increased number of pavement rehabilitation alternatives when the design life of the pavement has been achieved.

A program of regularly scheduled maintenance should be undertaken to preserve the integrity of the pavement structure. During pavement service life, heavy vehicle traffic should be limited to heavy duty pavement areas. Cracks in pavement should be sealed as soon as possible to prevent moisture infiltration into the pavement subgrade. Drainage paths should be maintained to allow the free flow of surface water away from the structure and pavements, such as through regular cleanouts in catch basins

10.0 TEMPORARY HAUL ROADS

The above noted pavement and slab sections are not intended to support a large volume of construction traffic; rather they are intended to provide a stable base for the intended end use (i.e. slab or pavement). The majority of construction traffic should be limited to travel on temporary haul roads. Where these haul roads coincide with future pavement or slab-on-grade areas, the haul roads may need to be provided with a temporary increased thickness of fill material to protect the integrity of the subgrade. A minimum 300 mm thickness of fill is usually spread before light construction traffic can proceed. In areas of heavy construction traffic, about 500 mm or more of fill material may be required to limit subgrade disturbance, the actual amount dependent on site conditions at the time of construction. Furthermore the granular structures noted in the report are intended to be completed in full, in this regard the granular sections are not intended to be left exposed/unfinished over the winter or spring months and should be paved or have concrete placed as soon as possible in order to avoid the risk of subgrade weakening and subsequent remedial repairs of the placed granular structure.

11.0 Final Site Grading, Surface Drainage, and Sub-Drainage

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 asphalt paved areas) to 3 percent (for gravel surfaced 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.

Subdrainage systems are required where structures include a basement or a crawlspace, such as where a structurally supported main floor slab is provided. 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 layers. 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.

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.

12.0 TESTING AND CONSTRUCTION MONITORING

The engineering design recommendations presented in this report are based on the assumptions that an adequate level of construction monitoring will be provided during construction, and that construction will be undertaken in accordance with all applicable codes and regulations. Construction should be performed according to generally accepted industry standards of care. An adequate level of construction monitoring is considered to be:

1. For earthworks:	*- Full-time monitoring and compaction testing.*
2. For deep foundations:	*- Design review and full-time monitoring during construction.*
3. For concrete construction:	*- Testing of plastic and hardened concrete in accordance with CSA A23.1 and A23.2.*

Amec Foster Wheeler can provide CSA-certified concrete testing services on request. Amec Foster Wheeler 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 Foster Wheeler 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.

13.0 CLOSURE

The findings and recommendations of this report were based on the results of field and laboratory investigations, combined with an interpolation of soil and groundwater conditions between test hole locations. If conditions are encountered that appear to be different from those shown by the test hole drilled at this site and described in this report, or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and adjusted, if necessary.

Soil conditions, by their nature, can be highly variable across a site. Although not encountered in this investigation, 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 was prepared exclusively for the City of Winnipeg and their agents for the proposed development as described in the report. The data and recommendations provided herein should not be used for any other purpose, or by any other parties, without review and advice from a qualified geotechnical engineer. Where additional development is proposed at the subject site, additional geotechnical investigation should be undertaken. The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. No other warranty, expressed or implied, is given.

Yours truly,
Amec Foster Wheeler Environment & Infrastructure
A Division of Amec Foster Wheeler Americas Limited

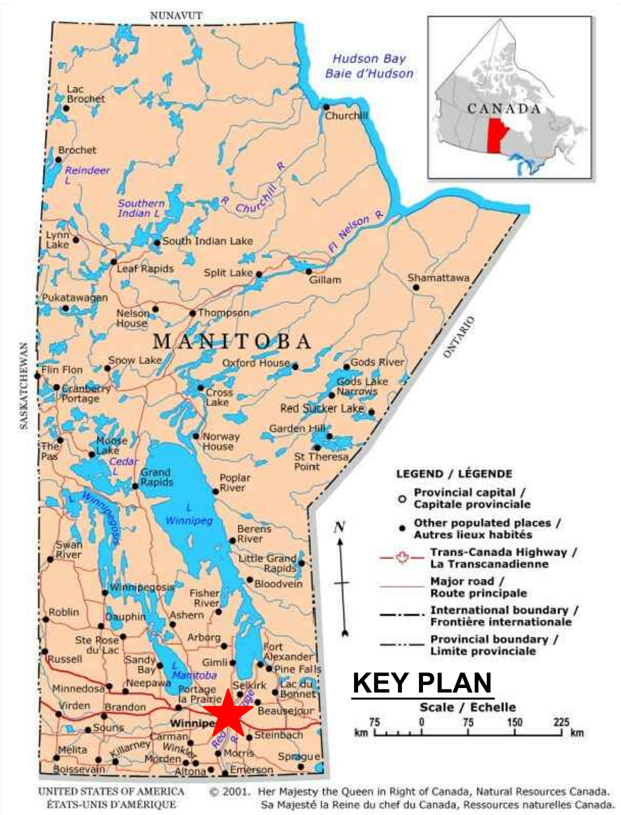
PROVINCE OF MANITOBA
T. R.
GLUCK
Member
22305
DEC 5/16
REGISTERED PROFESSIONAL ENGINEER

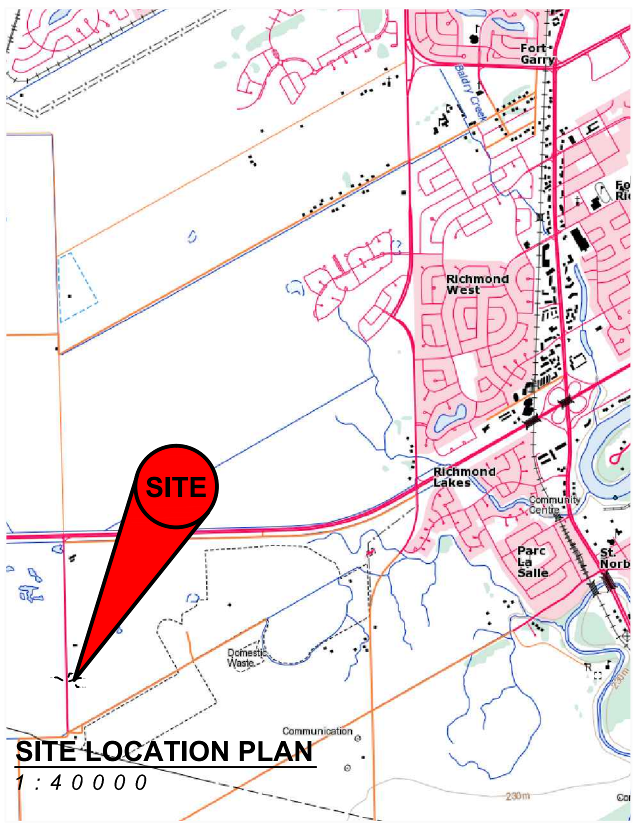
Trevor Gluck, P. Eng.
Associate Geotechnical Engineer

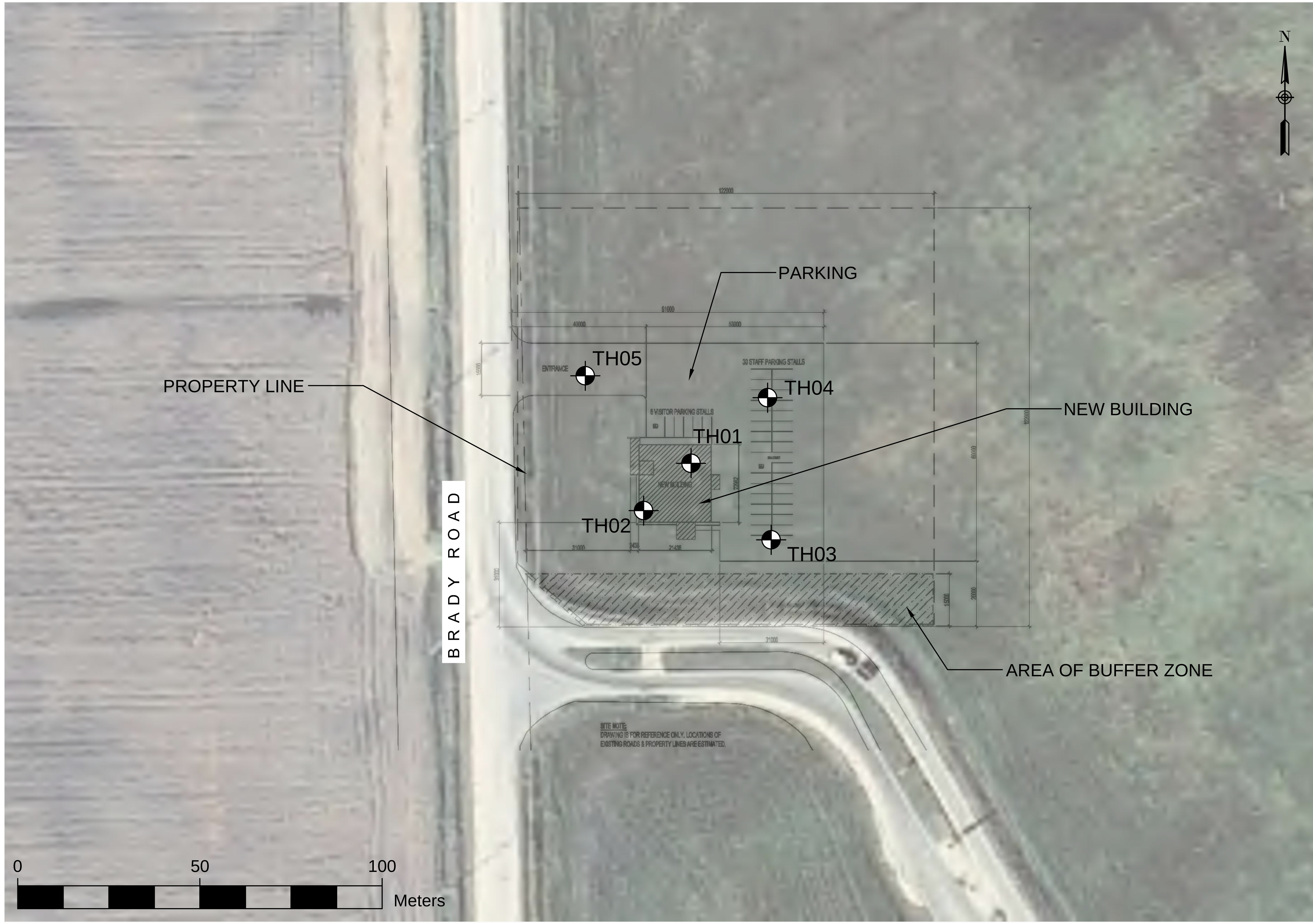
APEGM
Certificate of Authorization
Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited
No. 6017 Date: DEC 5/16

APPENDIX A

Figure 1: Test Hole Location Plan
Figures 2 to 6: Test Hole Logs







LEGEND:

TEST HOLE

REVISION	BY	DATE
----	----	----

CLIENT:

CITY OF WINNIPEG

Amec Foster Wheeler
Environment & Infrastructure
440 DOVERCOURT DRIVE
WINNIPEG, MANITOBA R3Y 1N4
PHONE: 204.488.2997 FAX:204.489.8261

amec foster wheeler

DWN BY: MD

CHK'D BY: TG

DATUM: NAD83

PROJECTION: UTM Zone 14 U

SCALE: AS SHOWN

GEOTECHNICAL INVESTIGATION
PROPOSED ADMINISTRATION BUILDING
BRADY LANDFILL
WINNIPEG, MANITOBA

SITE AND TEST HOLE LOCATION PLAN

DATE: DECEMBER 2016

PROJECT NO: WX18076

REV. NO.: ---

FIGURE NO: FIGURE 1

P:\JOBS\18000'S\18070'S\18076 – NEW BRADY ROAD LANDFILL ADMINISTRATION BUILDING – 1777 BRADY RD\DRAWINGS\WX18076.DWG

PROJECT: Administration Building - Brady Landfill	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH01**
CLIENT: City of Winnipeg	DRILL TYPE: Track Mounted Acker MP-5	PROJECT NO: WX18076
LOCATION: 1777 Brady Road	DRILL METHOD: 125mm Solid Stem Augers	ELEVATION:

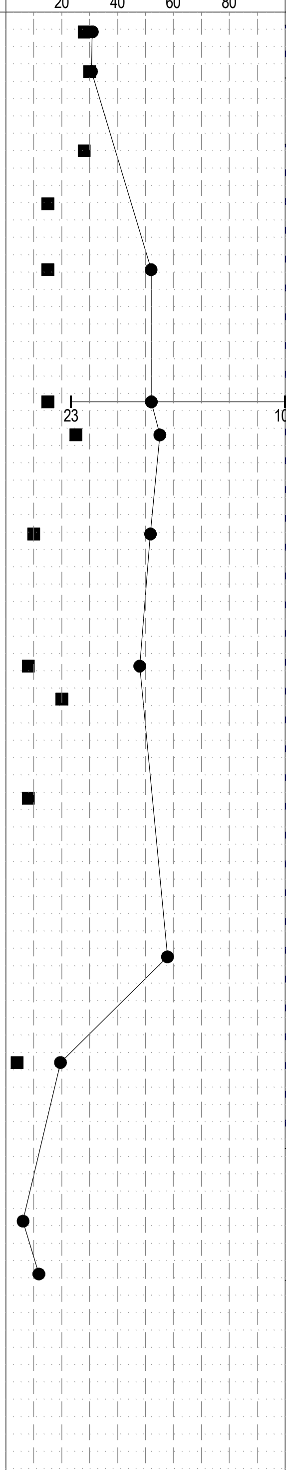
SAMPLE TYPE: Shelby Tube | No Recovery | SPT (N) | Grab Sample | Split-Pen | Core

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



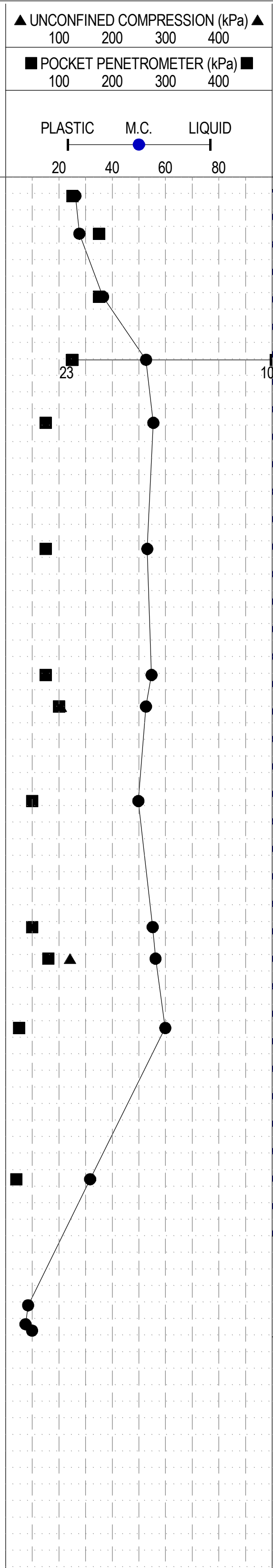
Depth (m)	MUSCS	SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
0	OH	ORGANIC CLAY - trace organics, silty, some sand, high plastic, moist, firm, dark grey to black, frequent rootlets	1		
	CH	CLAY - silty, trace gravel, high plastic, moist, stiff, brown	2		
1	ML	SILT - trace sand, low plastic, very moist, very soft, tan-brown			
	CH	CLAY - silty, trace gravel, high plastic, moist, firm to stiff, mottled grey-brown	3		
2		- occasional to frequent sulphate inclusions below 2.1m	4		
3		- occasional silt inclusions below 3.0m	5		
4			6		
5			7		
		- occasional oxidation inclusions below 5.2m			
6		- grey below 6.1m	8		
7		- firm below 7.0m	9		
8			10		
9			11		
10		- silt till layer 150mm thick between 10.5m and 10.7m	12		
11			13		
12		- soft below 11.7m	14		
13	ML	SILT (TILL) - sandy, some gravel, trace clay, moist, compact (inferred), tan to brown	15		
14		- damp below 13.4m - very dense below 13.7m	16	50/100	
			17		
15		AUGER REFUSAL AT 14.6m BELOW GRADE.			

NOTES:
- Slight sloughing was observed below 14.5m during drilling.
- Slight seepage was encountered between 0.8m and 1.5m and below 14.5m during drilling.
- Test hole remained open to 14.5m with no water above slough.
- Test hole was backfilled with auger cuttings and bentonite.

Amec Foster Wheeler Winnipeg, Manitoba	LOGGED BY: CM	COMPLETION DEPTH: 14.6 m
	REVIEWED BY: TG	COMPLETION DATE: 7 November 2016
	Figure No. 2	Page 1 of 1

PROJECT: Administration Building - Brady Landfill	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH02**
CLIENT: City of Winnipeg	DRILL TYPE: Track Mounted Acker MP-5	PROJECT NO: WX18076
LOCATION: 1777 Brady Road	DRILL METHOD: 125mm Solid Stem Augers	ELEVATION:

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	OH	CLAY - trace organics, silty, some sand, high plastic, moist, firm, dark grey to black, abundant rootlets	1		
	CH	CLAY - silty, high plastic, moist, stiff, brown, frequent silt lenses approximately 50 to 100mm thick	2		
1		- brown mottled grey, frequent silt and sulphate inclusions below 1.5m	3		
2		- firm below 2.4m	4		
3			5		
4			6		
6		- grey below 6.1m	7, 8		
7			9		
9			10, 11		
10		- firm to stiff below 9.8m	12		
11		- frequent silt till inclusions below 11.1m			
12			13		
13	ML	SILT (TILL) - sandy, some gravel, trace clay, moist, compact (inferred), tan to brown			
		- damp, very dense below 13.7m	14, 15, 16	50/100	
14		AUGER REFUSAL AT 14.0m BELOW GRADE.			

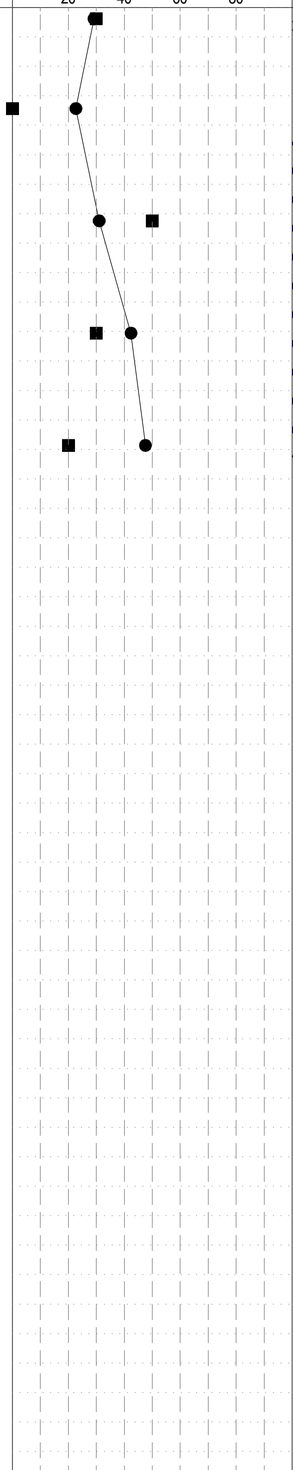
NOTES:
- No sloughing was observed during drilling.
- Slight seepage was encountered between below 13.1m during drilling.
- Test hole remained open to 14.0m with no recordable water level below grade upon completion of drilling.
- Test hole was backfilled with auger cuttings and bentonite.

Amec Foster Wheeler Winnipeg, Manitoba	LOGGED BY: CM	COMPLETION DEPTH: 14 m
	REVIEWED BY: TG	COMPLETION DATE: 7 November 2016
	Figure No. 3	Page 1 of 1

PROJECT: Administration Building - Brady Landfill	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH03**
CLIENT: City of Winnipeg	DRILL TYPE: Track Mounted Acker MP-5	PROJECT NO: WX18076
LOCATION: 1777 Brady Road	DRILL METHOD: 125mm Solid Stem Augers	ELEVATION:

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)	▲ UNCONFINED COMPRESSION (kPa) ▲ 100 200 300 400 / ■ POCKET PENETROMETER (kPa) ■ 100 200 300 400 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	Depth (m)
0			OH	CLAY - silty, trace organics, trace sand, high plastic, moist, stiff, dark grey, occasional rootlets	Grab	1			
			ML	SILT - trace sand, low plastic, very moist, very soft, tan-brown	Grab	2			
1			CH	CLAY - silty, high plastic, moist, very stiff, brown, frequent silt inclusions					1
				- stiff, brown mottled grey, occasional to frequent sulphate inclusions below 1.5m	Grab	3			
2					Grab	4			2
3					Grab	5			3
				TEST HOLE TERMINATED AT 3.1m BELOW GRADE. NOTES: - No sloughing or seepage was observed during drilling. - Test hole remained open to 3.1m and dry upon completion of drilling. - Test hole was backfilled with auger cuttings and bentonite.					
4									4
5									5
6									6
7									7
8									8
9									9
10									

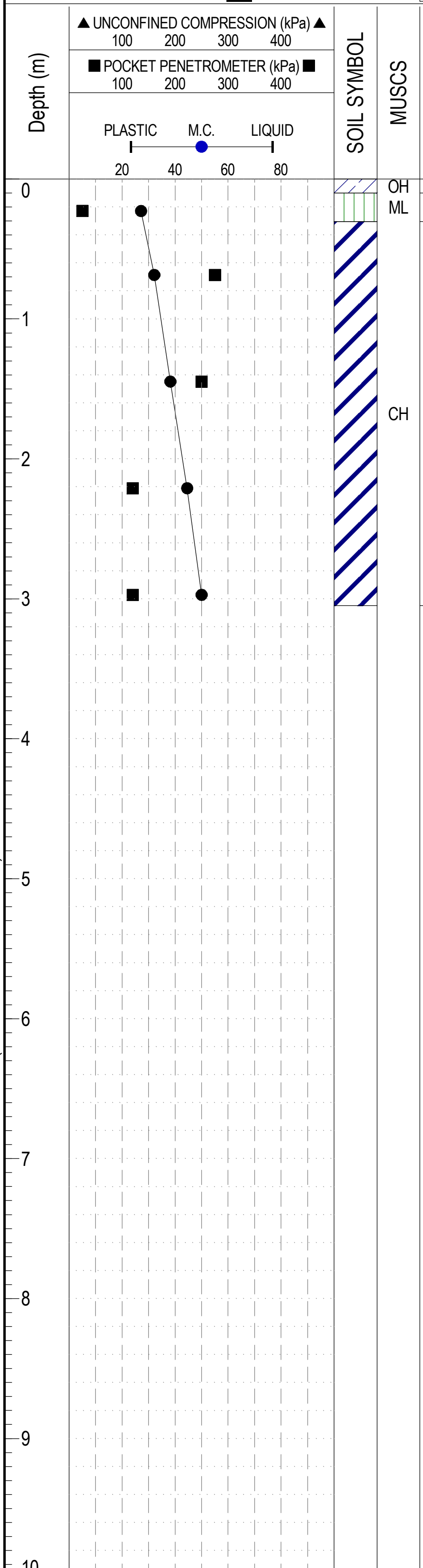


Amec Foster Wheeler Winnipeg, Manitoba	LOGGED BY: CM	COMPLETION DEPTH: 3.1 m
	REVIEWED BY: TG	COMPLETION DATE: 7 November 2016
	Figure No. 4	Page 1 of 1

PROJECT: Administration Building - Brady Landfill	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH04**
CLIENT: City of Winnipeg	DRILL TYPE: Track Mounted Acker MP-5	PROJECT NO: WX18076
LOCATION: 1777 Brady Road	DRILL METHOD: 125mm Solid Stem Augers	ELEVATION:

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

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



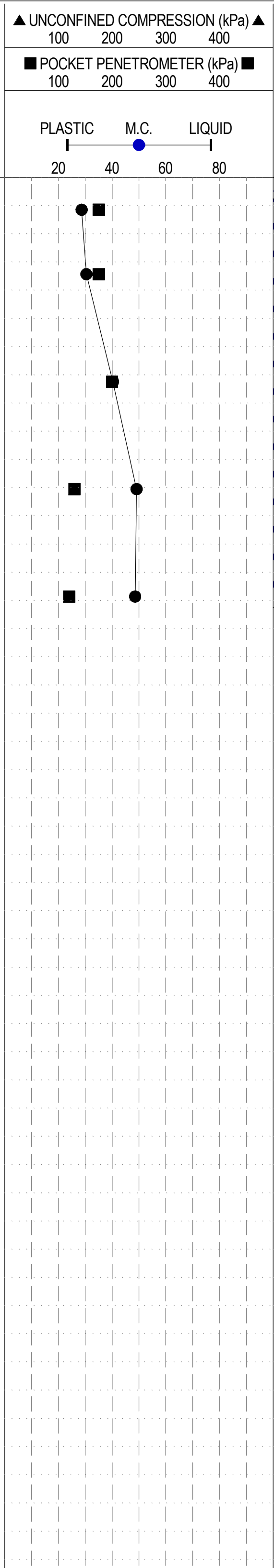
Depth (m)	MUSCS	SOIL DESCRIPTION	SAMPLE NO
0	OH	CLAY - silty, trace organics, trace sand, high plastic, firm, stiff, dark grey, occasional rootlets	1
	ML	SILT - trace sand, low plastic, very moist, very soft, tan-brown	
	CH	CLAY - silty, high plastic, moist, very stiff, brown	2
1		- brown mottled grey below 1.2m	3
		- stiff, frequent silt and sulphate inclusions below 1.5m	
2			4
3			5

TEST HOLE TERMINATED AT 3.1m BELOW GRADE.
NOTES:
- No sloughing or seepage was observed during drilling.
- Test hole remained open to 3.1m and dry upon completion of drilling.
- Test hole was backfilled with auger cuttings and bentonite.

PROJECT: Administration Building - Brady Landfill	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH05**
CLIENT: City of Winnipeg	DRILL TYPE: Track Mounted Acker MP-5	PROJECT NO: WX18076
LOCATION: 1777 Brady Road	DRILL METHOD: 125mm Solid Stem Augers	ELEVATION:

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)	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	Depth (m)
0		OH	CLAY - silty, trace organics, trace sand, high plastic, moist, firm, dark grey to black	Grab	1			
			CLAY - silty, high plastic, moist, very stiff, brown, frequent silt inclusions	Grab	2			
1				Grab	3			1
		CH	- stiff, brown mottled grey, occasional to frequent sulphate inclusions below 1.5m					
2				Grab	4			2
3				Grab	5			3
			TEST HOLE TERMINATED AT 3.1m BELOW GRADE. NOTES: - No sloughing or seepage was observed during drilling. - Test hole remained open to 3.1m and dry upon completion of drilling. - Test hole was backfilled with auger cuttings and bentonite.					
4								4
5								5
6								6
7								7
8								8
9								9
10								

WX18076 - ADMINISTRATION BUILDING - BRADY LANDFILL.GPJ 16/12/05 03:04 PM (GEOTECHNICAL REVISED)

amec foster wheeler	**Amec Foster Wheeler** **Winnipeg, Manitoba**	LOGGED BY: CM	COMPLETION DEPTH: 3.1 m
		REVIEWED BY: TG	COMPLETION DATE: 7 November 2016
		Figure No. 6	Page 1 of 1