GEOTECHNICAL INVESTIGATION
McDERMOT AVENUE OUTFALL UPGRADES
WINNIPEG, MANITOBA

Submitted to:
MMM Group Limited
111-93 Lombard Avenue
Winnipeg, Manitoba
R3B 3B1

Attention: Mr. Edmund Ho, P. Eng.

Submitted by:
AMEC Environment & Infrastructure
A Division of AMEC Americas Limited
440 Dovercourt Drive
Winnipeg, Manitoba
R3Y 1N4

12 November 2013

AMEC File No. WX17253

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1.0 INTRODUCTION

As authorized by Mr. Jim Lukashenko of the MMM Group Limited, AMEC Environment and Infrastructure, a Division of AMEC Americas Limited (AMEC), has completed a geotechnical investigation for the proposed McDermot Avenue outfall chamber upgrades, located to the east of the intersection of McDermot Avenue and Riverfront Drive in Winnipeg, Manitoba.

The scope of work for the geotechnical investigation was conducted in accordance with AMEC proposal number WPG2012.316, dated 19 July 2013. The purpose of the geotechnical investigation was to investigate the subsurface conditions at the site in order to provide geotechnical recommendations necessary for the design and construction of the outfall structure, as well as to evaluate the riverbank stability at the site.

The following report summarizes the field and laboratory testing programs, describes the subsurface conditions encountered at the test hole location, and presents geotechnical engineering recommendations for design and construction of the gate chamber at the McDermot Avenue outfall.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Project Description

Based on the information provided, it is understood that the existing outfall was constructed in 1966 and is serviced by a positive gate housed in a buried chamber approximately 60 meters west of the west property line on Ship Street. It is understood that the current gate is inoperable, and during times of high river elevations water backs up into the storm relief system, reducing capacity of the system and potentially leading to the need for additional flood protection.

As such, it is understood that the McDermot Avenue Outfall Upgrade project will consist of construction of a new gate chamber structure on the east boulevard of the intersection of McDermot Avenue and Waterfront Drive, which is located approximately 80 m from the normal summer water level of the Red River. The new chamber will include a positive gate with electric actuator and a flap gate, which is to provide greater operational control of the outfall system. In addition, the project includes the removal of the existing positive gate, installation of a submersible pump with buried discharge piping leading to the storm sewer on McDermot Avenue, construction of a removable weir immediately downstream of the existing gate chamber, connections to electrical supply, miscellaneous ladders and hatches, the installation of a new manhole chamber upstream of the gate, restoration of the ground surface, construction and landscape services. It is understood that no changes will be required at the outlet of the structure to the river.

Based on the understanding of the project, it is understood that the proposed new gate chamber will have a footing with dimensions of approximately 0.5 m thick and 9.9 m x 5.4 m in plan. The proposed footing will be situated beneath the existing 2.7 m diameter concrete drainage pipe at approximate geodetic elevation 220.4 m.

It was also understood that the proposed concrete gate chamber will have the following approximate design loads:

Table 1: Summary of New Gate Chamber Loading Conditions

Loading Conditions	Unfactored Loads (kN)
Empty Chamber	6,700
Chamber Filled with Water	9,650

2.2 Site Description

The proposed development will be situated in Stephen Juba Park, to the east of the intersection of McDermot Avenue and Waterfront Drive on the riverbank of Red River as illustrated in Figure 1. The proposed new gate chamber will be situated several metres east of the concrete curb of the Waterfront Drive in the garden area. It is expected that some trees with size of 100 mm to 300 mm diameter will need to be cut for allowing the development on site.

In general, the topography of the riverbank has a benched ground surface with a relatively flat ground of about 4 m at the crest followed by slopes varying from 3H: 1V near the top of the back to 8H: 1V closer to the river with a flat section about 45 m wide near the river. The bank slopes at approximately 6H: 1V down to the river channel. The cross section of the riverbank along the existing drainage pipe is illustrated in Figure 2.

Visual observations of the riverbank did not identify any signs of active (or recent) slope movements. Riprap erosion protection was observed along the river's edge and around the existing outfall discharge.

3.0 FIELD AND LABORATORY PROGRAMS

3.1 Field Investigation

Prior to initiating drilling, AMEC notified public utility providers (i.e. Manitoba Hydro, MTS, Shaw, 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 also retained the services of a private utility locator to identify the locations of any City's owned utilities in the work area.

On 24 September 2012, AMEC supervised the drilling of one test hole (TH01) on a full time basis at the approximate location illustrated in Figure 3. It should be noted that as a result of overhead trees branches, the test hole was drilled approximately 8 m northeast of the proposed gate chamber and was situated at the south edge of the asphalt paved bicycle path.

The test hole was advanced using a truck mounted CME drill rig equipped with 125 mm diameter solid stem augers, owned and operated by Subterranean (Manitoba) of Winnipeg, Manitoba. Test hole TH01 was drilled to practical auger refusal within the very dense glacial silt till at about 15.7 m below the existing ground surface.

During drilling, AMEC field personnel visually classified the soil stratigraphy within the test holes in accordance with the Modified Unified Soil Classification System (MUSCS). Any observed seepage and/or sloughing conditions were recorded as drilling progressed and upon completion of drilling. Grab samples were collected from each test hole at selected depths and retained in sealed plastic bags for shipping, review, and testing in AMEC's Winnipeg laboratory. The relative consistency and the undrained shear strength of the cohesive soils encountered were evaluated in the test hole using a hand held Pocket Penetrometer (PP). Readings were noted at regular intervals during drilling. A relatively undisturbed Shelby tube sample was collected in the clay for laboratory strength testing. One Standard Penetration Tests (SPT) was also conducted within the silt till and split spoon samples were retrieved upon completion of test hole.

The sloughing conditions and the depth to groundwater within the test hole were measured as drilling progressed and immediately after removal of the augers from the test hole. Subsequently, a 50 mm diameter standpipe piezometer was installed and the test hole was backfilled with bentonite, auger cuttings, and silica sand as shown in the test hole log.

A detailed test hole log summarizing the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole location is presented in Figure A1 of Appendix A. Actual depths noted on the test hole log may vary by ± 0.3 m from those recorded due to the drilling method and the method by which the soil cuttings are returned to the surface. Summaries of the terms and symbols used on the test hole logs and of the Modified Unified Soil Classification System are also presented in Appendix A.

3.2 Laboratory Testing

A laboratory testing program was carried out on selected soil samples obtained from the test hole, and consisted of the following:

- Moisture content determinations
- Unconfined compressive strength test
- Hydrometer Test (to determine the soil grain size distribution)
- Atterberg Limits (to determine the soil plasticity)

Laboratory test results are summarized on the test hole log in Figure A1.

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

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

- Various Fills
- Alluvial Medium Plastic Clay
- Glacial Silt Till

Various Fills

Generally, the subsurface soil consisted of various sand, silt and clay fill layers to about 5.5 m below grade at the test hole location.

Sand fill was encountered at the surface of the test hole to 0.5 m below grade. The sand was silty, poorly graded, loose (inferred), damp, grayish brown and contained a trace amount of clay. Clay fill was present below the sand fill and extended to about 5.5 m below grade. The clay fill was generally silty, medium to high plastic, moist, stiff, mottled dark grey and greyish brown and contained trace gravel, trace to some sand, trace wood and occasional glass pieces.

A layer of silt fill was interbedded in the clay fill from 3.2 m to 3.8 m below grade. The silt fill was clayey, low to medium plastic, moist, stiff and light brownish grey.

Alluvial Medium Plastic Clay

Alluvial medium plastic clay was encountered below the fill and extended from 5.5 m to 14.8 m below ground surface. The alluvial medium plastic clay was silty, moist, firm to stiff, grey and contained some sand. Frequent sand lenses and layers were present throughout the alluvial clay. The interbedded sand layers were wet, poorly graded, fine grained, loose to compact (inferred), grey to dark grey and contained varied clay and silt contents.

The clay moisture content varied from approximately 27% to 38% with an average of 30%. The result of the hydrometer test determined that the sample contained about 2-14% sand, 60-66% silt and 26-33% clay particles. The clay sample tested had plastic and liquid limits of 20-22% and 44-48%, respectively. At about 10.3 m below the existing ground surface, the unconfined compressive strength of the clay was determined to be approximately 40 kPa at 4.8 % strain.

Glacial Silt Till

Glacial silt till was encountered below the alluvial clay and extended to the depth explored at 15.7 m below grade. The glacial silt till was clayey, low plastic, very moist to wet, compact to dense (inferred), light greyish brown and contained trace gravel in the top 0.4 m and then became damp to moist and dense.

A detailed description of the soil profile encountered at the test hole can be found on the test hole log in Figure A1 or Appendix A.

4.2 Groundwater and Seepage Conditions

Seepage and sloughing conditions were noted during drilling in the test hole. The depth to slough and accumulated water level within the test hole was measured within about 10 to 15 minutes after completion of drilling. Water levels were also measured in the standpipe on 10 October 2013, 16 days after drilling. Details of the groundwater conditions, as well as the observed sloughing, are summarized in Table 2.

Table 2: Test Hole Exploration Depths and Groundwater Conditions at TH01

Test Hole	Test Hole Depth (m)	On Completion of Drilling		Groundwater Level (m) Below Grade Monitored on 10 Oct 2013	Test Hole Ground Surface Elevation (m) asl
		Depth to Slough (m)	Depth to Ground water (m)		
TH01	15.7	11.6	11.6	4.3	229.59

Notes:

- 50mm diameter standpipe was installed to 12.2m below grade with screen section from 6.1 to 12.2m in the alluvial clay.

Based on the groundwater monitoring data collected to date and presented in Table 2, the piezometric surface in the clay layer appears to be approximately 4.3 m below the existing ground surface in October 2013. Due to frequent sand lenses and layers in the alluvial clay, which are relatively permeable in nature, the groundwater condition in the alluvial clay may likely be influenced by the river and/ or the bedrock aquifer particularly at deeper depth. It should be noted that the water levels will vary on a seasonal and annual basis.

5.0 GEOTECHNICAL RECOMMENDATIONS

5.1 General Evaluation

It is understood from MMM that the gate chamber foundation will most likely comprise a cast-in-place concrete footing bearing within the clay. On this basis, the following sections provide discussion and recommendations as they pertain to design and construction of the proposed gate chamber, specifically: “allowable” bearing pressure, lateral earth pressures on chamber walls; temporary construction dewatering requirements; and foundation concrete type.

5.2 Excavation Stability

5.2.1 Shoring

Based on the depth of the gate chamber, the soil conditions encountered and the proximity of Waterfront Drive, shoring or some other form of excavation support will likely be required to maintain excavation stability for construction of the gate chamber. Currently, it is envisaged that suitable excavation support systems would consist of one of the following:

- A braced sheet piled walls inclusive of lateral struts such that lateral support is provided to all four sides of the excavation; or
- A soldier piles system with timber lagging.

Excavations that are extended to a depth of about 9.8 m or deeper will be subject to groundwater issues as the groundwater table was determined to be about 225.3 m asl when monitored in early October 2013, which is 4.9 m above the base of the proposed excavation. Furthermore, the frequent sand layers suggest the water levels will likely be tied to that of the river level and therefore significantly higher water levels could occur during river flood stages. Therefore, seepage from the side walls and the base of the excavation should be anticipated and a dewatering system, either internally and/or externally, is likely to be needed for the project.

Generally, there are three criteria for design of a supported excavation as follows:

1. The stability at the base of the excavation against shear failure should have a factor of safety (FS) greater than 2.0
2. The stability at the base of the excavation against piping from water seepage has to be greater than 2.0
3. To protect against base heave the porewater pressure at the base of the excavation should not exceed 70% of the total stress at this point.

The following sections discuss each of the above noted design considerations.

5.2.1 Base Stability Against Shear Failure

As stated earlier, the stability of the excavation base against shear failure has to be evaluated to confirm a safe excavation base condition. The failure mechanism occurs from inadequate resistance of the loads imposed by the differences in grades inside and outside of the excavation. Based the current excavation size of 5.4 m wide and 10 m long and about 9.8 m deep, the FS against shear failure is determined to be about 1.3. As a result, based on Canadian Foundation Engineer Manual (CFEM), if the FS against shear failure is less than 1.5, then the depth of penetration of the support system must extend below the base of the excavation. In this case, the proposed soldier piles and timber lagging system will not be feasible. A shoring system that consists of sheet piles would be acceptable in the case and therefore recommendations for a soldier pile and timber lagging system will not be provided.

5.2.2 Minimum Recommended Sheet Piling Embedment Depth due to Seepage

Based on a groundwater elevation of about 225.3 m asl (i.e. about 4.9 m below the existing grade of the proposed gate chamber footprint monitored in Oct 2013) and an excavation depth of 220.4 m asl (i.e. about 9.8 m deep), any excavations will be subjected to significant seepage and sloughing conditions. The seepage rate will depend on the actual soil conditions at the excavation, including those at the base. Depending on the construction period, significantly higher water levels could also be encountered (i.e. during spring or summer flood events).

Given the conditions the base of the excavation may be vulnerable to piping, heave or boiling. This issue can "generally" be mitigated by driving the sheet piles below the proposed base of the excavation, thereby reducing the hydraulic exit gradient to a condition lower than the critical hydraulic exit gradient. The depth of sheet pile embedment required to satisfy the basal stability condition may be determined by following the method provided in Section 22.3.2.1 of the Canadian Foundation Manual (CFEM), 4th Edition, depending on the shape of the proposed excavation (i.e. either 1. Long and Rectangular, 2. Circular, or 3. Square) as presented in Figure 4. For this method, the calculated exit gradient, i_{exit}, must be less than the critical gradient, $i_{critical}$, divided by a suitable factor of safety. That is;

$$i_{exit} < i_{critical} / FS$$

$$i_{exit} = C \times \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$

Where:

- i_{exit} = Calculated Exit Gradient
- $i_{critical}$ = Critical Exit Gradient; taken as 0.83
- FS = Factor of Safety; taken as 2.0
- C = Constant; taken as 1.0 for rectangular, 1.3 for circular and 1.7 for square excavation configurations
- h = height of the groundwater within the clay above the excavation base.
- b = one half the excavation width; in meters (Due to a rectangular shape excavation, the b should be taken as the longer side of its dimension, such as the 10 m in this case)
- φ_1 = Obtained from Figure 4.
- φ_2 = Obtained from Figure 4
- d_1 = depth from the groundwater table to the base of the sheet pile; in meters
- d_2 = depth of the base of the sheet pile below the excavation base; in meters
- T_1 = depth from groundwater table to an impervious layer below the depth of the excavation at depth; Assuming the

impervious layer as the bedrock layer at approximately 213.60 m asl

- T_2 = depth from the excavation base to an impervious layer at depth; Assuming the impervious layer as the bedrock layer at approximately 213.60 m asl

It should be noted that the value of 'h' provided above is determined based on groundwater monitoring results determined from monitoring conducted on 10 October 2013. Increases in groundwater elevation can occur due to heavy rains, rises in the nearby river level, or rises in the underlying bedrock aquifer which may be connected to the glacial till layer. As a result, AMEC recommends that groundwater conditions be monitored prior to and during construction to verify the basal stability of the excavation and the design sheet pile penetration depth. As well, the construction period should be reviewed so that the likelihood of higher water levels during construction can be determined.

The above method for exit gradient assessment will allow determination whether basal stability will be of concern for a given groundwater condition and/ or the length of sheet piles. Dewatering will be needed if the required sheet length is not achievable. Based on the current groundwater condition encountered on site at 225.3 m asl, which is about 4.9 m above the proposed excavation level at 220.4 m asl, AMEC has determined that the even where the sheet piles are installed into the glacial silt till, there is a potential risk of basal instability occuring. As a result, groundwater control such as dewatering of the existing groundwater level is needed to improve the basal instability. The details of construction dewatering are discussed in Section 5.2.5.

5.2.3 Soils Heave at the Excavation Base

In addition to the exit gradient assessment, the porewater pressure at the tip of the sheet piles should not exceed 70% of the total vertical stress of the soils between the excavation base and the tip of the sheet piles. If this condition cannot be achieved for the given sheet pile penetration, then a greater penetration depth will be required. The total vertical stress of the soils can be calculated using the unit weight of soils that are presented in Table 3, times the total thickness of the soils.

5.2.4 Lateral Earth Pressure for Temporary Shoring

The distribution of lateral earth pressure on a shoring system depends on many factors including, but not limited to, the soil type, groundwater conditions over the depth of the shoring, surcharge loading at the surface, rigidity of the system, and the target degree of shoring wall movement resulting in full, or partial, development of active earth pressures.

Based on the premise that the shoring will consist of steel sheet piles or soldier piles with timber lagging systems that are braced internally with a system of steel walers and/or struts in order to restrain shoring movements, the 'apparent' distribution of earth pressure to be resisted by a braced shoring system in the layered soils should be calculated according to Section 26.10.3, Braced Retaining Structures – Loading Conditions of CFEM, 4th Edition, page 409, utilizing the apparent earth pressure distributions shown in Figure d and the following soil parameters. The Figure d in the CFEM is presented in Figure 5 of the report.

Table 3: Lateral Earth Pressure Coefficients on the Gate Chamber Walls

Soils Parameter	Various Fills	Alluvial Medium Plastic Clay	Glacial Silt Till
ϕ', Internal Friction Angle	17	27	40
γ_t, total unit weight (kN/m^3)	18	18.5	21
γ', submerged unit weight (kN/m^3)	7.2	8.2	11.2
"At-rest" Earth Pressure Coefficient, K_o	0.71	0.55	0.36
"Active" Earth Pressure Coefficient, K_A	0.55	0.38	0.22
"Passive" Earth Pressure Coefficient, K_p	1.49	1.89	2.63

In generally, the lateral earth pressures is calculated as follow,

$$\sigma_h = K \times \sigma_v$$

where,

K = Earth Pressure Coefficient
σ_v = Total Vertical Stresses ($\gamma \times H_o$)
H_o = Embedment Depth of Wall Below Grade (m)

This lateral earth pressures will then be applied to the Figure 5 for lateral stress assessment while designing the struts (braced supports).

The passive resistance is developed by that portion of the sheet or soldier pile below excavation grade. In the case of soldier piles and lagging, the passive resistance should be taken to act on the diameter of the embedded portion of the soldier pile below the lowest excavation grade. A safety factor of at least 2.0 should be applied to the passive resistance calculations.

Total unit weights of the soils should be used above the water table. A combination of submerged soil unit weights and static water head pressure should be used below the water table. The design depth of the water table should be established on the basis of monitoring data over a period of time leading up to the design of the shoring system. Prior to the temporary shoring construction, the groundwater conditions should be monitored to confirm the estimations/ assumptions made during the design phase are still valid. If the water table rises to an elevation higher than those estimated in the design phase, the entire shoring system should be evaluated to confirm whether the design remains appropriate.

The value of K used in the equation above will be influenced by the amount of lateral wall movement that is considered permissible.

a) If moderate wall movements (i.e. 1.0% to 2.0% of the excavation depth) can be permitted, the pressure may be computed using the coefficient of active earth pressure, K_a.

b) If services adjacent to the excavation exist at a shallow depth, at a distance less than H (height of the wall) behind the top of the wall, and not closer than 0.5 H and some movements (i.e. 0.3% to 1.2% of the excavation depth) of services can be tolerated, the pressure may be calculated using a coefficient determined as follows:

$$K = 0.5(K_a + K_o)$$

c) If services exist at a shallow depth, or if there are adjacent existing foundations at a distance less than 0.5 H behind the top of the wall or if movements of services are intolerable, the pressure should be computed using the coefficient of earth pressure at rest, K_o.

AMEC can provide the lateral earth pressures distributions of the proposed shoring system once the details of the excavation and shoring type are finalized.

5.2.5 Surcharge Loads

In addition to earth pressures, lateral stresses generated by any applicable surcharge loads also need to be evaluated in the design. The surcharge considered should include the effects of loads from street traffic, construction equipment, and any other loads that may be transferred to the walls of the excavation during the construction period.

For line or point surcharge loads, the lateral pressures should be determined using the relationships given in Figure 6. 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.

5.2.6 Construction Dewatering

As mentioned in the previous section, the need for on site construction dewatering should be- anticipated. High groundwater flows, either through the base of the excavation or through voids in the interlocking sheet piles or the timber lagging, could lead to loss of ground resulting in reduced excavation stability.

Construction dewatering can generally be performed by pumping the water from inside and/or outside of the excavation. Generally, pumping of water from outside of the excavation is a safer approach than pumping the water from inside of the excavation. An external dewatering system may consist of the installation of perimeter dewatering wells surrounding the excavation. Prior to implementation of the external dewatering system, a pump test is highly recommended to determine the permeability of the insitu ground and to evaluate the effectively of a potential external dewatering system. Typically, the design and operation of the dewatering system would be the responsibility of the construction contractor, with review and approvals from the engineering design team.

It should be noted that the groundwater level inside of the excavation should be kept at a minimum of 1 m below the base of the excavation for allowing a clean and dry subgrade. In addition, even where an external dewatering system is implemented, there may be potential of slight water seeping into the excavation, if this occurred, or where redundancy is needed an internal dewatering system (pumping water inside of the excavation) should also be implemented. A temporary dewatering measure should be used to control any potential of water flow into the excavation to preserve the stability of the excavation and reduce the potential for groundwater accumulation within the excavation. The internal dewatering system may be comprised of collection trenches/ pits and sump pits, with appropriate filtering.

Due to potential water issues, a temporary shoring system that consists of tightly spaced or interlocked pile walls systems, such as the steel sheet piles will be of advantage.

It is expected that the bearing surface will consist of silty and sandy medium plastic clay or sand. As a result, the bearing surface may be susceptible to disturbance, particularly when it is wet with high groundwater condition. In this regard, avoiding disturbance of the bearing surface is vital. Protection of the bearing surface may be achieved with the placement of a lean-mix concrete slab (or mudslab) directly on the bearing subgrade. Pressure relief ports through the mudslab, and/or some form of dewatering below the base of the mudslab may be necessary to mitigate potential build-up of hydrostatic forces on the base of the slab.

Groundwater discharge should meet the necessary local government requirements for water quality and should be designed to facilitate sampling if and where required. In this regard, where fine particles are collected within the groundwater, it may be necessary to remove the fines (i.e. by) prior to disposal in City storm sewers. This may require the use of silt curtains, sedimentation or filtering to contain suspended water-born particles and limit sediment transport during discharge. Furthermore, the loss of fine particles may be an indication of a more serious concern regarding the potential for piping. Therefore, the loss of ground both from the excavation base and from behind the shoring should be monitored during construction. It is recommended that the condition of the base excavation be evaluated by AMEC during construction to determine the effectiveness of the external and internal dewatering system as well as assess the subgrade bearing surface.

5.2.7 Excavation Staging

All shoring members (i.e. struts, walers, timber lagging, sheet piles, soldier piles and etc) should be designed and checked or all stages of partial and full excavation.

5.2.8 Shoring Wall Monitoring

Shoring performance and general condition of the excavation should be monitored both during and following construction of the shoring wall. The shoring wall should be regularly monitored for ground loss and the presence of voids behind the shoring, particularly where seepage is encountered during excavation. All voids detected should be immediately backfilled with sand and/or grout. Shoring monitoring should include measurement of lateral and vertical movement of shoring walls, settlement monitoring of hard surfaced areas around the site as applicable, and measurement of vertical movements of the excavation base.

For sheet piled walls, the lateral wall movement is anticipated to be less than two (2) percent of the excavation depth throughout all stages of construction, although movements will depend on the rigidity of the design, as the lateral wall movement of the sheet piled walls is a function of the relative stiffness of the sheet piles and the spacing of the lateral support (i.e. struts). Movements are also depending on the workmanship, and how quickly the lateral support can be provided during the excavation. These movements will generally be smaller if the horizontal supports are installed as soon as the support level is reached. Similarly, vertical settlement of surface grades within a horizontal distance of the shoring equal to three times the depth of the excavation and is anticipated to be less than one (1) percent of the depth of excavation if construction is in keeping with best practices. AMEC can provide further guidance on the excavation movements, once the detail of the shoring design is finalized. If greater lateral movements or vertical settlements are observed, the design and construction of the shoring system should be reviewed.

5.2.9 Other Considerations

It should be noted that there are additional issues that should also be considered for a temporary shoring system is used for this application as follows:

1. The removal of the sheet piles, soldier piles, wood lagging, etc after construction will create voids in the soil behind the walls of the gate chamber. All voids should be properly backfilled with either granular fill, compacted in place by water jetting, or using a cement grout. The choice of backfill material should take into account designs for both horizontal stresses and frost effects pertinent to the specific backfill type selected. As an alternative, voids may be eliminated by casting the gate chamber walls directly against the steel sheet piles and leaving the steel sheets in place permanently, if sheet piles were utilized.
2. The construction of the proposed gate chamber is favourable to be held in the winter when there is reduced chance of elevated water levels

5.3 Gate Chamber Foundation

5.3.1 Design Footing Bearing Pressure

It is understood that the proposed outfall gate chamber will have a concrete footing bearing at a depth of approximately 0.6 m below the existing 2700 mm diameter concrete drainage pipe. The depth of the pipe at the gate chamber location is estimated to be at approximately 9.8 m below existing ground surface of the proposed gate chamber location (i.e. 9.2 m below grade at the test hole location), at approximately 220.4 m asl. In addition, the proposed footing will be 5.4 m wide and 10 m long.

On this basis, the ultimate bearing capacity of the alluvial clay is determined to be 340 kPa. A geotechnical resistance factor of 0.5 should be utilized under the limit state design approach. As a result, the proposed footing will have a factored geotechnical resistance of 170 kPa.

It is recommended that where a bearing pressure of 150 kPa is used for design of the chamber foundation total settlement to be less than 25 mm and this could be considered the serviceability limit state. According to the information provided to AMEC, it was understood that the proposed footing may be designed to consist of a factored bearing pressure of 170 kPa. Total settlement of the footing under a 170 kPa service load is estimated to be between 40 and 50 mm. It is further cautioned that additional settlement could occur where disturbance and/or softening of the subgrade occurs during construction. The expected settlement should be reviewed and where the amount of settlement is tolerable, the serviceability limit state can be modified accordingly.

The bearing surface of the gate chamber should be excavated in a manner to minimize disturbance of the subgrade. The bearing surface should be trimmed free of softened or loose soil, kept free of water, and protected from any other environmental effects that will cause disturbance to the subgrade condition (such as frost).

5.3.2 Buoyancy

According to the project design criteria, which states that the proposed gate chamber is to be designed to meet a 1:700 year flood level at 230.31 m asl, the proposed gate chamber should be designed against an uplift pressure due to buoyancy under a flood level of 230.31 m asl. This assumes that the soils below the gate chamber are hydraulically connected to the river.

Resistance to buoyancy will be provided by the dead weight of the gate chamber and soil friction along the exterior sidewalls of the gate chamber. The allowable side friction resistance along the perimeter walls of the gate chamber between the soil and the concrete may be taken as 11 kPa between depths of 2.4 and 9.8 m below the existing grade.

5.3.3 Lateral Earth Pressures on Buried Gate Chamber Walls

The permanent walls of a buried concrete gate chamber will be required to resist lateral earth pressures and hydrostatic pressure from the surrounding soil and groundwater. Where the gate chamber is cast directly against the temporary shoring or where backfill placed against the wall of the chamber is lightly compacted, the lateral soil pressure (*p*) distribution may be assumed to be trapezoidal in shape and increase linearly with depth as illustrated on Figure 7.

Lightly to moderately compacted backfill typically corresponds to soils placed and compacted to between 93 percent and 95 percent of standard Proctor maximum dry density (SPMDD). Settlements under the self weight of such compacted backfill is dependent on the soil type used, however usually do not exceed 2 percent of the fill height. In cases where the backfill is well to highly compacted, settlements will be less, however, the additional lateral pressures induced on the wall by compaction must also be considered in the design of the below grade walls. AMEC can provide lateral earth pressure distributions for highly compactive backfill upon request.

The design of the gate chamber wall should also take into account the hydrostatic component acting on the wall. The groundwater levels considered in design of the subsurface walls may be taken as 230.31 m asl (i.e. 700 year flood level).

It is anticipated that a braced excavation will be formed against the face of the excavation, and as such, limited relaxation of the retained soils will occur. As such, the use of the 'at-rest' lateral earth pressure coefficient K_o in the design of unyielding gate chamber walls is recommended. The 'at-rest' earth pressure coefficient is presented in Table 3 in Section 5.2.4.

It is recommended that a cap of clay, concrete or asphalt should be placed at or just below the ground surface adjacent to the foundation walls to reduce the migration of surface water into the underlying granular backfill materials. If a clay cap is used, the clay cap should have a minimum thickness of approximately 0.3 m and should extend a minimum of 3 m horizontally from the gate chamber walls.

5.3.4 Frost Considerations

Based on local experience, the maximum frost penetration depth of 2.4 m is expected at the site without snow cover. Frozen ground could impose uplift force to the gate chamber due to the adfreeze bond between the frozen soils and the gate chamber walls. Adfreeze bond stress is typically in the range of 65 kPa between the frozen fine-grained soils to concrete.

Resistance to the adfreeze stress would be provided through by the combined mass of the gate chamber structure plus frictional resistance of the soil in contact with the concrete walls below the depth of frost. The allowable frictional resistance between the soil and the concrete may be taken as 11 kPa between depths of 2.4 and 9.8 m below the existing grade of the proposed gate chamber footprint. Alternatively, the effect of adfreeze can be reduced through the application of a bond breaker around the perimeter of the chamber within the depth of frost. A suitable bond breaker may consist of a Dow Ethafoam product or a smooth geosynthetic liner material fixed to the exterior of the chamber walls.

However, notwithstanding the above, the gate chamber will extend through the zone of frost penetration. Portions of the gate chamber located within the depth of frost penetration must be structurally designed to resist increased lateral pressures induced by frost. 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].

It should be noted that uplift force due to frost is not an additive from buoyancy as both have different mechanisms and each occur at different time of the season.

5.4 Riverbank Slope Evaluation

5.4.1 Slope Stability Criteria

The project site is located on a slight outside bend of the west bank of the Red River in Winnipeg. Since the site is located within about 100 m of the Red River, the proposed works will require securing a Waterway Permit from the office of the Riverbank Management Engineer of the City of Winnipeg in accordance with the City of Winnipeg Waterway By-law 5888/92. In order to successfully obtain a Waterway Permit for this work, it will be necessary to illustrate that the proposed works will not negatively impact the riverbank, or the river flow regime in any way and that the proposed works are situated at a suitable offset from the river such that they are not in jeopardy of becoming damaged due to potential riverbank movements.

Pursuant to Clause 4.3 of the Waterway By-law, *"a permit shall not be issued for work to be done in a regulated area unless the [applicant] demonstrates to the reasonable satisfaction of the Director that the proposed work will not, or will not have a tendency to:*

- *a) restrict or impede surface or sub-surface water flow;*
- *b) endanger the stability of any land, including the bed of a waterway;*
- *c) cause land to slip into a waterway; or*
- d) *adversely alter the channel of a waterway."*

5.4.2 Slope Stability Evaluation

Given the nature of the gate chamber, clauses 4.3 a) and d) are inherently satisfied.

In order to verify that the proposed outfall structure meets clauses 4.3 b) and c), slope stability modeling of the existing riverbank stability was completed. Traditionally, local design practice and philosophy employed in geotechnical evaluation of structures within the regulated waterways where the offset (or setback) of a structure is specified is to evaluate the factor of safety of the riverbank against an adopted minimum target factor of safety (FS) of 1.5 under 'normal' conditions, and against a minimum target FS of 1.3 under 'extreme' design conditions. Where the factor of safety of the offset meets or exceeds both of these criteria, no additional

[1] As per Canadian Foundation Engineering Manual, 3rd Edition, P. 429, an earth pressure coefficient K=1 should be used in combination with insulation for highly frost susceptible soils.

stabilization works are required. If the factor of safety at the offset of the structures fails one or both of these criteria, slope stabilization works may be required.

Slope stability analyses were completed on a single cross- section taken through the riverbank at the location of the gate chamber using the slope stability software package, Slope/W, produced by Geo-Slope International of Calgary, Alberta. Specifically, the factor of safety of circular slip surfaces was estimated using the grid and radius method and the Morgenstern-Price method with a half sine variation of inter-slice forces. The topography of the cross-section was developed based on the topographic survey completed by the City of Winnipeg, while the soil stratigraphy was developed based on the test hole log and assumed uniform conditions extending to the river.

5.4.3 Topography

As stated above, the topography of the riverbank along the existing drainage pipe was developed by the City of Winnipeg as presented in Figure 2. The ground surface profile was developed along the drainage pipe alignment, which was parallel to McDermot Avenue and at an angle to the Red River. Accordingly, the applicable cross section is not along this alignment, rather it is along a line perpendicular to the river. As a result, the riverbank surface profile that was established by the City of Winnipeg was altered to represent the riverbank cross section profile that is perpendicular to the river channel. This modified riverbank cross section profile has been utilized as the basis of the numerical modeling for the riverbank stability assessment

5.4.4 Soil Conditions

It should be noted that advanced geotechnical laboratory testing (i.e. Triaxial and Direct Shear Tests) was beyond the scope of the riverbank assessment conducted by AMEC. In this regard, the effective shear strength parameters used for the soil strata observed in test hole TH01 were assumed based on commonly used strength parameters for similar observed soils in Manitoba, plus the experience from AMEC’s previous projects.

Visual evidence of previous riverbank movements such as tension cracks, slumps, soil rotation, failure scarps and samples containing slickensided surfaces, etc. was not observed. Given the lack of evidence of previous slope movement, the use of fully softened or post peak effective shear strength parameters was considered to be appropriate for the clay soils encountered at this site. These parameters are summarized in Table 4. The post peak strength of a soil is the strength condition that resides between the maximum (i.e. peak) and the minimum (i.e. residual) possible values. The maximum or peak condition is present where soils are intact and have not undergone straining. The minimum or residual condition occurs where the soils have undergone significant straining associated with large scale riverbank failure. The post peak strength condition develops when straining within the soil mass goes beyond that of the maximum strength condition, causing the soil to become weaker, but not so far as to cause a failure condition to exist. The post peak condition does, however, take into account the potential for an overall fissured soil structure to exist. The use of post peak strengths is a common modeling approach for the shear strengths of soils in close proximity to riverbanks and is generally a conservative assumption.

Table 4: Summary of Average Isotropic Shear Strength Parameters

Parameter	Fills	Clay	Glacial Silt Till
Unit Weight, γ (kN/m^3)	18	18.5	Bedrock[1]
Effective Post Peak Shear Strength Parameters	ϕ' = 17° c' = 1 kPa	ϕ' = 27° c' = 1 kPa	
Notes: [1.] Glacial Silt Till was modelled as an impenetrable surface, allowing the use of both circular and composite slip surfaces typical of observed riverbank failures in Winnipeg.			

5.4.5 Red River Levels

The proposed Site is adjacent (i.e. about 300 m) to the City of Winnipeg's James Avenue river level monitoring station. As such, the river levels in the model were taken to be equivalent to those recorded in the James Avenue monitoring station. The historical river levels at the James Avenue monitoring station are illustrated in Figure 8.

River levels fluctuate through different stages that, depending on the combination with groundwater levels, significantly influence the stability of a river bank. In Winnipeg, during summer, the Red River Normal Summer Water Level (NSWL) is controlled at St. Andrews Lock and Dam at Lockport, 20 km downstream of Winnipeg. This structure regulates the level of the Red River generally at or about Elev. 223.8 m asl in The City of Winnipeg near James Avenue. The NSWL level is maintained in the city for recreation purposes following the spring runoff event until the latter part of October of each year, at which point the river level is allowed to return to its Normal Winter Ice Level (NWIL) over a period of a number of weeks.

Sporadically and within the seasonal fluctuations of the Red River, there are occasions where extreme high and low water conditions may also occur that again, depending on the combination with groundwater levels, may also largely influence the stability of a riverbank. Such events usually occur during either spring flood when the river can be well above the normal flooding levels or during winter when the Red River is returned to uncontrolled levels and reaches levels that can be below normal winter ice levels. Based on information provided to AMEC, it is understood that the proposed Site is to be designed to a 700 year flood condition, which corresponds to a river level of 230.31 m asl. As a result, the stability of the riverbank at the proposed outfall gate chamber under this extreme condition has to been taken into consideration.

Based on the above discussion, design river elevations at the Site are as follows:

a) Normal Summer Water Level condition (NSWL): 223.8 m;
b) Extreme Winter Ice Level (EWIL): 221.6 m; and
c) Extreme Spring Water Level condition (ESWL): 230.31 m asl (700 year flood)

5.4.6 Groundwater Conditions

Groundwater levels at the site are expected to be influenced by interconnections between the river, the underlying carbonate aquifer and the overburden alluvial soils, as well as surface water infiltration in a lesser degree.

Typically, the groundwater levels within the overburden soils vary between summer, winter, and flood induced peak conditions. During spring flooding conditions, the overburden groundwater levels rise in response to the increase in river elevation, seasonal runoff and carbonate aquifer levels. At this time, the groundwater elevation is typically below the river level, the amount of which cannot be easily predicted as it depends on the specific riverbank conditions, the permeability of the soils and the duration of the flood event. As spring draws to a close and river level recedes towards the normal summer level, the groundwater level within the soils near the face of the bank generally remain elevated above the river level for a period of time depending on the soil conditions at the specific location. During summer, the river level and the groundwater level in both the overburden soils and the bedrock vary to some degree; however, generally achieve a typical or ‘normal’ condition. As summer draws to a close and river levels draw down to the winter level, the groundwater level within the overburden soil remain elevated above the winter river level for some period of time, again depending on the permeability of the soils.

These variations in groundwater and river levels give rise to variations in riverbank stability, generally with the most critical conditions occurring either during late fall to early winter when the river level is low and the riverbank groundwater conditions remain elevated above the river level or immediately following the spring flood event as the river level recedes to the normal summer condition, but the groundwater conditions in the riverbank remain elevated. The normal summer condition is typically taken to occur during summer when both the river level and the riverbank groundwater conditions are relatively stable.

Monitoring instrumentation to measure groundwater levels was not installed at the site; however, based on AMEC’s past experience, the groundwater levels are expected to be around 1.5 m to 2.5 m higher than the river levels depending on the time of the year and the seasonal conditions.

Groundwater monitoring on the 50 mm diameter standpipe obtained from the test hole location on 10 October 2013 in the alluvial clay and the glacial silt till was determined to have piezometric level of about 225.3 m (i.e. 4.9 m below the existing grade of gate chamber footprint). Based on AMEC's experience for groundwater conditions in Winnipeg near the riverbank, the groundwater levels could be +/- 2.0 m from the monitored 225.3 m in Oct 2013.

5.4.7 Stability Modeling Approach & Results

Slope Stability Assessment on Existing Riverbank for Various Scenarios:

Given the river levels interpreted above and the groundwater monitoring data collected on site the following three (3) conditions have been modelled for the purpose of evaluating the existing slope conditions:

1. Normal Summer Design Conditions – These conditions represent the coupled river level and groundwater conditions that are presumed to "regularly" occur at the NSWL. This condition is based on the interpreted NSWL together with the existing groundwater monitoring data collected near the proposed location of the gate chamber. The groundwater elevation near the crest of the riverbank was determined roughly by following the contour of the riverbank slope to intersect the design river level.

    - River NSWL = 223.8 m asl
    - Groundwater Elevation in clay at chamber = 227.3 m asl (2.9 m above water level monitored in Oct 2013 assuming the water level in the spring will be 2m high than those monitored in Oct 2013)
    - Groundwater Elevation in clay at riverbank crest = 223.8 m asl (assume 1.0m below grade)

2. Spring Drawdown Extreme Design Conditions – These conditions were selected to reflect the extreme condition that is presumed to occur following rapid drawdown from the ESWL to the NSWL. The effect of the drawdown was evaluated by using seepage modeling program (Seep/W) to estimate the porewater pressure in the riverbank. In the program, a transient seepage analysis was utilized taking the ESWL (i.e. 230.31 m) reached and then the river level quickly receded to the NSWL (i.e. 223.8 m) over a period of three months.

3. Fall Drawdown Extreme Design Conditions – These conditions were selected to reflect the extreme condition that is presumed to occur following drawdown from the NSWL to the EWIL. The effect of the drawdown was evaluated by using seepage modeling program (Seep/W) to estimate the porewater pressure in the riverbank. In the program, a transient seepage analysis was utilized taking the NSWL (i.e. 223.8 m) and then the estimated that the river level receded to the EWIL (i.e. 221.6 m) over a period of one month that is typically observed from the historical river level trend. This groundwater condition is considered to be conservative for this case given that an extremely low river level in the fall is unlikely to occur if the groundwater level leading into the fall event was above conditions that would typically be expected.

It should be noted that the stability assessment performed herein focuses only on the riverbank stability at the location where the gate chamber will be constructed. Localized or shallow slip surfaces having lower factors of safety may occur downslope of the riverbank crest; however, these slip surfaces are not considered to be relevant to the proposed gate chamber development.

From the above three scenarios, factors of safety (FS) of 2.91, 2.63 and 2.58, were determined for Condition 1, 2 and 3, respectively. These results all exceed the minimum FS requirements identified in Section 5.4.2 for the extreme and normal conditions at the proposed gate chamber location. The results of these scenarios are presented in Figure 9 to 11.

Slope Stability Assessment on Riverbank for the in-place Gate Chamber:

Further to the existing riverbank stability assessment, a scenario was created to evaluate the stability of the riverbank following installation of the proposed gate chamber. It is assumed that a tension crack may develop at the downslope side of the installed gate chamber and that the crack would be filled with water, which would cause hydrostatic pressures acting on the soil wall. The groundwater and river conditions were assumed to be similar to Scenario 2, Spring Drawdown Extreme Conditions that has generated the lower safety factor on the existing riverbank stability prior to any development.

Under the above conditions, a FS of 2.62 was calculated and the result is illustrated in Figure 12. The result indicates that even there is tension crack at the chamber wall, the FS of the river over a global stability with failure plane towards and into the river will still be consistent with the finding observed in Scenario 2 stated above. This result suggests that under the worst case scenario, the proposed gate chamber would not negatively impact the riverbank stability.

Based on the results above, and notwithstanding any potential site grading, stockpiling of excavated soil during construction, and/or other construction sequencing, the proposed gate chamber will not:

a) endanger the stability of any land, including the bed of a waterway; or
b) cause land to slip into the waterway.

5.4.8 Erosion Protection

Based on site reconnaissance performed on site, it was observed that the existing outfall consisted of riprap protection. Given the presence of the erosion protection, further work is not considered necessary.

5.4.9 General Guidelines for Maintaining Slope Stability

The following general guidelines are recommended for maintaining the existing stability of the riverbank:

- All existing vegetation along the riverbank should be maintained in its existing conditions.
- In general, existing grades should not be modified as a result of current or future construction on site. Where changes to the existing grades are proposed, either temporary (i.e. soil stock pile during construction) or permanently (i.e. the final design grade is higher than the current grade), AMEC should be contacted to evaluate the riverbank stability pertaining to these conditions.

5.5 Foundation Concrete Type

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 CSA standard A23.1-04, 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 AMEC's experience in Winnipeg, 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.

5.6 Testing and Monitoring

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 excavation:
  - monitor the groundwater conditions prior to construction.
  - evaluate the excavation base after completion of excavation to assess the basal stability and seepage conditions for dewatering assessment
  - monitor the installation of sheet piles
  - monitor vertical and horizontal shoring movements
- for foundations:
  - design review and review of the bearing surface prior to placement of concrete.
- for concrete construction:
  - testing of plastic and hardened concrete in accordance with CSA A23.1-04 and A23.2-04.
  - 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 gate chamber to confirm that the geotechnical recommendations have been correctly interpreted. AMEC further requests the opportunity to review the soil and groundwater conditions encountered as excavation proceed so that the assumptions made in preparing this report can either be confirmed, or so that recommendations provided in this report can be modified to reflect such different conditions as are encountered.

The contractor should be advised that it is anticipated that the geotechnical engineer will not be on site on a full-time basis. Therefore, the timely reporting by contractor staff of unusual events such as, but not limited to, loss of ground, changes in soil behaviour, movements of roadway surfaces and shoring, and changes in dewatering volumes will be very important in ensuring a suitably rapid response to potentially serious circumstances.

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 herein for design of the proposed McDermot Avenue Outfall Upgrades are based on a geotechnical evaluation of the findings in the geotechnical test hole drilled at the site. If conditions are encountered that appear to be different from those shown in the test hole log and described in this report, or if the assumptions stated herein are not in keeping with the design, AMEC should be notified and given the opportunity to review the current recommendations in light of any new findings. 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.

Soil conditions, by their nature, can be highly variable across a construction site. The placement of fill during and prior to construction activities on a site can contribute to variable soil conditions. A contingency amount should be included in the construction budget to allow for the possibility of variations in soil conditions, which may result in modification of the design, and/or changes in construction procedures.

This report has been prepared for the exclusive use of MMM Group Limited, and their design agents, for specific application to the development described within 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.

The findings and recommendations of this report have been prepared in accordance with generally accepted soil and foundation engineering practices. No other warranty is made, either expressed or implied.

Respectfully submitted,

AMEC Environmental & Infrastructure
a division of AMEC Americas Limited

Reviewed By:

Wing- Keat Wong, M. Eng., P. Eng.
Geotechnical Engineer

Harley Pankratz, P. Eng.
Senior Associate Geotechnical Engineer
V.P.: Eastern Prairies and Northern Alberta

APPENDIX A

Explanation of Terms and Symbols
Figure A1: Test Hole Log (TH01)

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	w	Natural Moisture Content (ASTM D2216)
	and hydrometer test	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	γ	Unit weight of soil or rock
pp	Pocket penetrometer strength	γ_d	Dry unit weight of soil or rock
*q	Triaxial compression test	ρ	Density of soil or rock
q_u	Unconfined compressive strength	ρ_d	Dry Density of soil or rock
*SB	Shearbox test	C_u	Undrained shear strength
SO_4	Concentration of water-soluble sulphate	→	Seepage
		▼	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	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

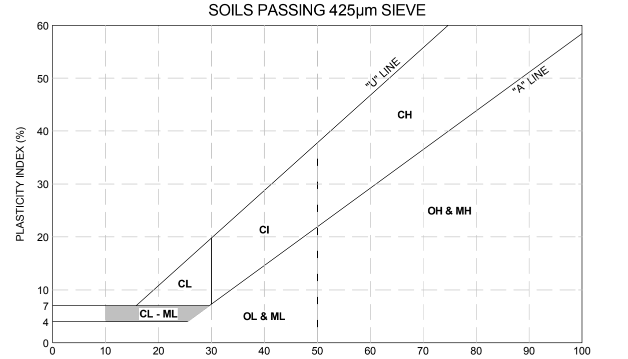
ROUNDED OR SUBROUNDED:

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.

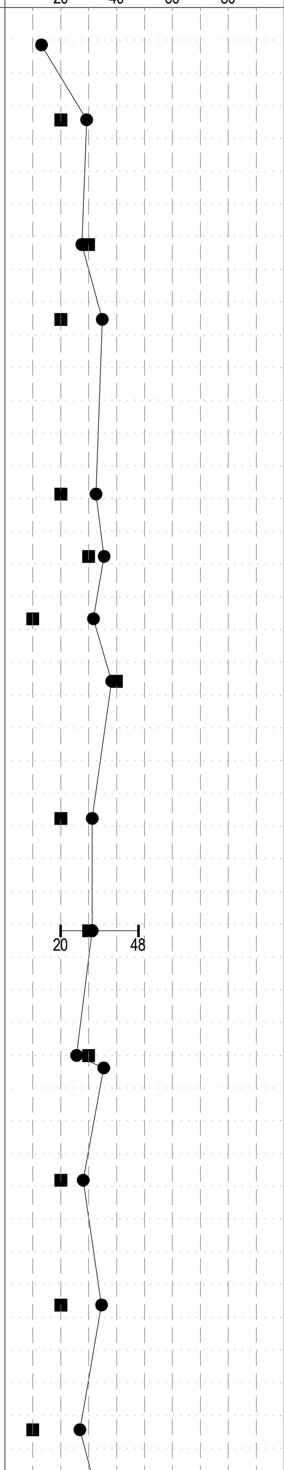
AMEC Environment & Infrastructure
a Division of AMEC Americas Limited

PROJECT: McDermot Ave. Gate Chamber	DRILLED BY: Subterranean (Manitoba) Ltd.	BORE HOLE NO: **TH01**
CLIENT: MMM Group Inc	DRILL TYPE: CME	PROJECT NO: WX17253
LOCATION: Winnipeg, Manitoba	DRILL METHOD: 125mm SSA	ELEVATION: 229.59 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)	SOIL SYMBOL	MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	COMMENTS	ELEVATION (m)
0		SM	SAND (FILL) - silty, trace clay, poorly graded, loose (inferred), damp, greyish brown	Grab	1			
		CI-CH	CLAY (FILL) - silty, some sand to sandy, trace gravel, high plastic, damp to moist, stiff, mottled grey and brown	Grab	2			229
1				Grab	3			228
2			- moist, trace sand below 1.8m	Grab	4			227
			- occasional silt inclusions below 2.7m					
3			- sandy, medium plastic, stiff, dark greyish brown, trace wood pieces below 3.1m	Grab	5			
		ML	SILT (FILL) - clayey, low to medium plastic, moist, stiff, light brownish grey	Shelby Tube / Grab	6 / 7			226
4		CH	CLAY (FILL) - silty, some sand, trace gravel, high plastic, moist, stiff, dark greyish brown	Shelby Tube	8		Groundwater measured at 4.3m below grade Oct 10, 2013	225
5			- trace glass pieces (~3cm long), trace wood chips encountered below 4.6m	Shelby Tube	9			224
		CI-CH	CLAY (ALLUVIAL) - silty, trace to some sand, meidum to high plastic, moist, stiff, grey to dark grey, occasional sandy silt lenses (<2mm thick) - sandy below 5.8m	Shelby Tube	10		Hydrometer Analysis Results @ "5.3m": Gravel= 0.0% Sand= 1.6% Hydrometer Analysis Results @ "5.8m": Clay= 32.6%	
6		SC	SAND - clayey, silty poorly graded, fine grained, very moist to wet, loose (inferred), light grey to grey, mm of clay laminations	Shelby Tube / Grab	11 / 12			223
7		CI	CLAY (ALLUVIAL) - silty, sandy, meidum plastic, moist to very moist, stiff, grey to dark grey, trace roots (<1cm dia), organic ordour	Shelby Tube	13			222
		SC	SAND - clayey, silty, poorly graded, fine grained, wet, very loose (inferred), grey to dark grey					
8		CI	CLAY (ALLUVIAL) - silty, sandy, medium plastic, moist, firm to stiff, dark grey - frequent sand lenses (mm) below 8.2m	Shelby Tube	14			221
9			- firm, very moist below 8.8m	Shelby Tube	15			

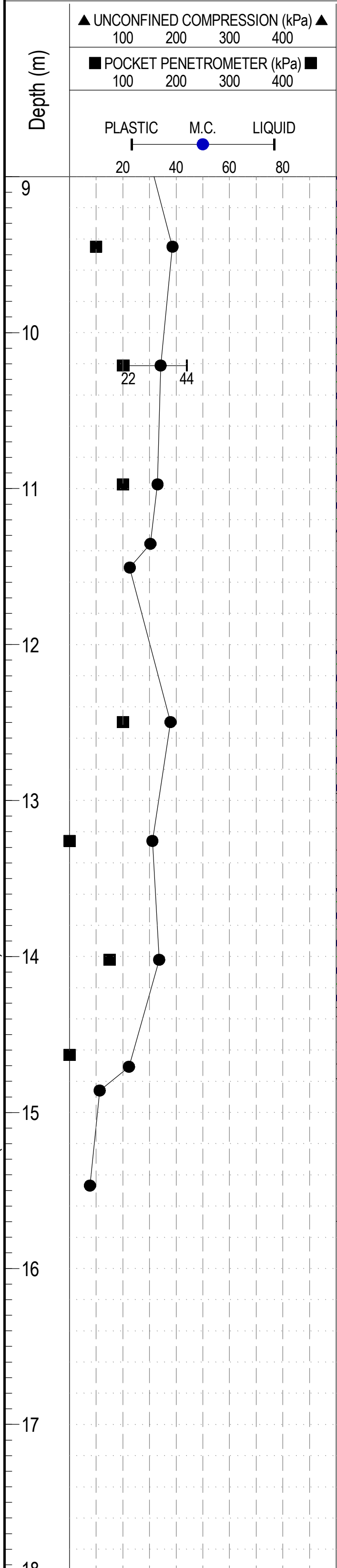
17253 - MCDERMOT AVE OUTFALL UPGRADE.GPJ 13/11/08 05:01 PM (GEOTECHNICAL REVISED)

amec	AMEC Environment & Infrastructure Winnipeg, Manitoba	LOGGED BY: JW	COMPLETION DEPTH: 15.7 m
		REVIEWED BY: WKW	COMPLETION DATE: 24 September 2013
		Figure No. 1	

PROJECT: McDermot Ave. Gate Chamber	DRILLED BY: Subterranean (Manitoba) Ltd.	BORE HOLE NO: **TH01**
CLIENT: MMM Group Inc	DRILL TYPE: CME	PROJECT NO: WX17253
LOCATION: Winnipeg, Manitoba	DRILL METHOD: 125mm SSA	ELEVATION: 229.59 m

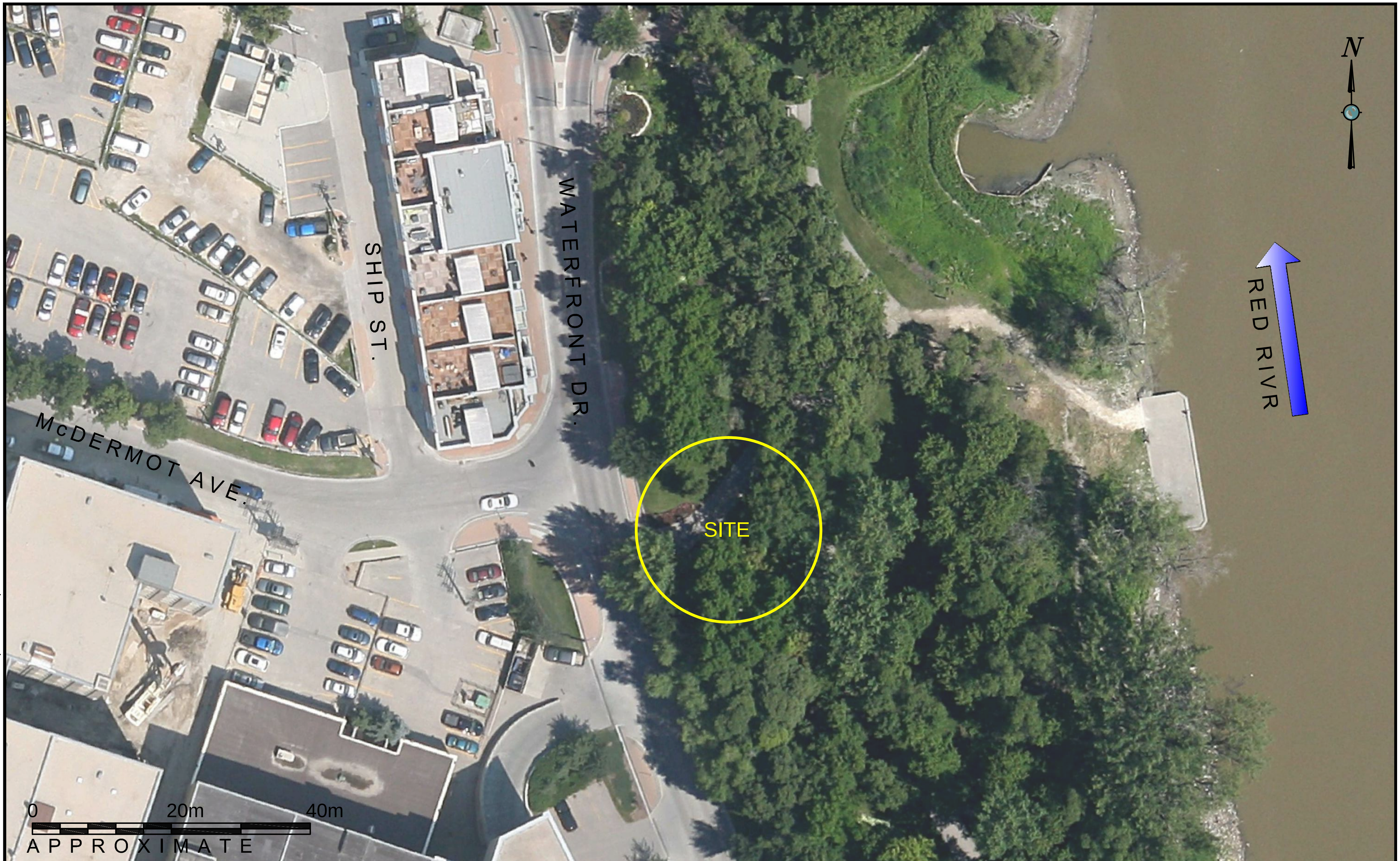
SAMPLE TYPE: Shelby Tube, No Recovery, SPT (N), Grab Sample, Split-Pen, Core

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



Depth (m)	SOIL SYMBOL / MUSCS	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NO	SPT (N)	SLOTTED PIEZOMETER	COMMENTS	ELEVATION (m)
9 – 11.3	CI	- 200 mm thick sand lense encountered at 9.9m	Shelby Tube	16, 17, 18		Sand / slotted	Hydrometer Analysis Results @ "9.9m": Gravel= 0.0% Sand= 14.1% Silt= 59.7% Clay= 26.2% Unconfined Compressive Test at 10.3m Max Compressive Stress = 39.9 kPa at 4.8% strain Unit Weight = 1853 kg/m^3	220, 219, 218
11.3 – 12.0	SP	SAND - trace silt, poorly graded, fine grained, wet, loose (inferred), grey to dark grey	Grab Sample	19				
12.0 – 13.0	CI-CH	CLAY (ALLUVIAL) - silty, sandy, medium plastic, moist, firm to stiff, grey to dark grey, frequent silty sand lenses (<1cm thick)	Shelby Tube	20		Drill Cuttings		217
13.0 – 13.6	SP	SAND - trace silt, poorly graded, fine grained, wet, loose (inferred), grey to dark grey, occasional clay lenses (< 2mm thick), organic odour	Shelby Tube	21				216
13.6 – 14.4	CI	CLAY (ALLUVIAL) - silty, sandy, medium plastic, moist to very moist, firm to stiff, grey to dark grey, frequent silty sand lenses (< 3mm thick)	Shelby Tube	22				
14.4 – 14.8	SP	SAND - trace silt, trace gravel, poorly graded, fine grained, wet, compact (inferred), grey to dark grey	Shelby Tube / Grab Sample	23, 24				215
14.8 – 15.7	ML	SILT (TILL) - sandy, clayey, trace gravel, low plastic, very moist to wet, dense (inferred), light greyish brown - some clay, damp, dense below 15.2m	Grab Sample / SPT (N)	25, 26	81			214
15.7 – 18		AUGER REFUSAL AT 15.7m ON SUSPECTED BOULDER OR BEDROCK Notes: - Slight seepage from 6.4m to 6.9m and from 11.3m to 12.0m observed upon completion of drilling. - Significant sloughing observed upon completion of drilling. - Test hole remain open to 11.6m for 10minutes upon completion of drilling. - One 50mm diameter standpipe installed on completion of drilling.						213, 212

APPENDIX B

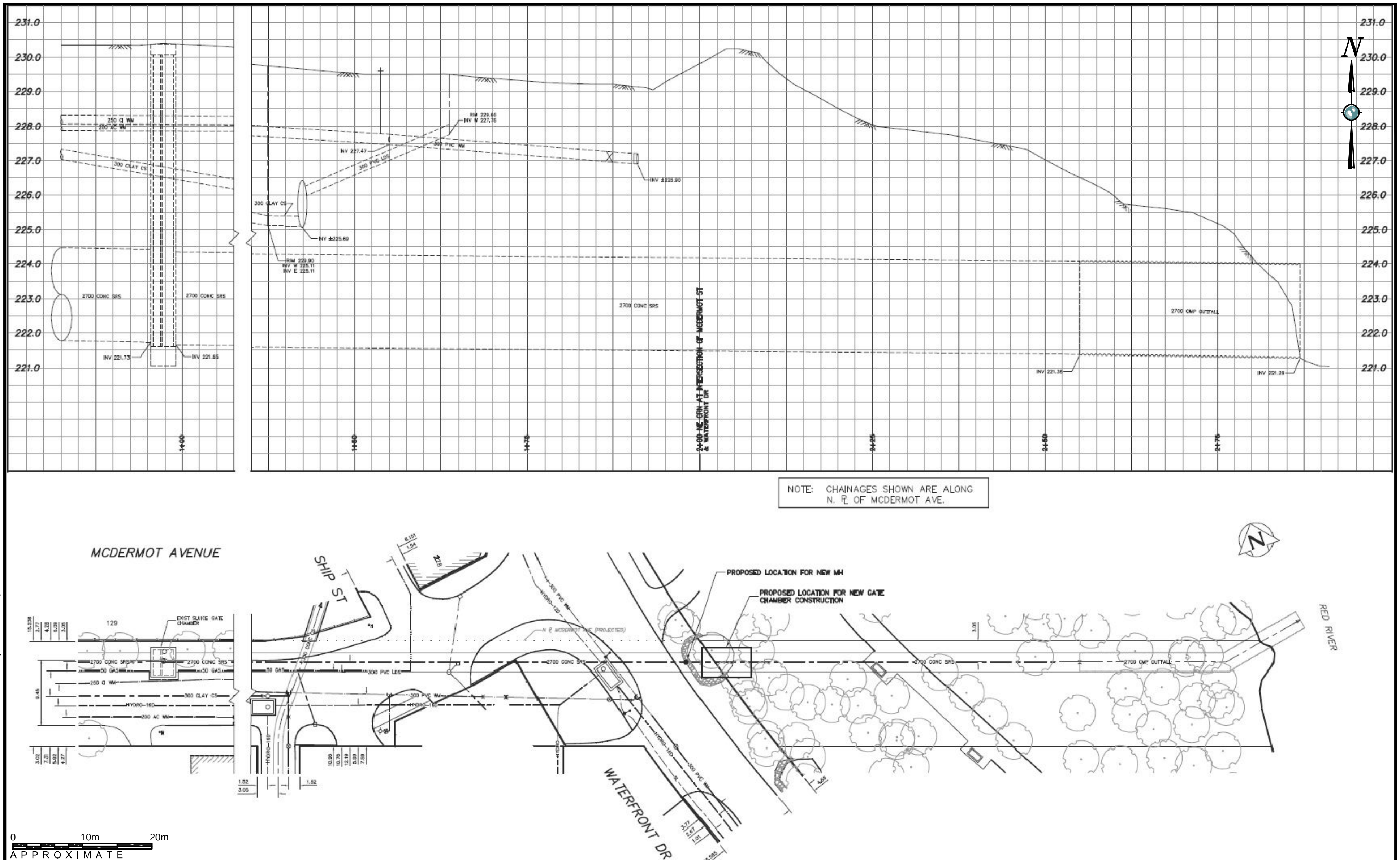


P:\JOBS\17200'S\17250'S\17253 – MCDERMOT AVE OUTFALL UPGRADE\DRAWINGS\WX17253.DWG

CLIENT LOGO: MMM GROUP	CLIENT: MMM GROUP LIMITED	DWN BY: MD	REV. NO.: A
		CHK'D BY: WKW	DATE: NOVEMBER 2013
AMEC Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	amec	DATUM: NAD83	PROJECT NO: WX17253
		PROJECTION: UTM Zone 14 U	FIGURE No. FIGURE 1
		SCALE: AS SHOWN	

GEOTECHNICAL INVESTIGATION
MCDERMOT AVENUE OUTFALL CHAMBER UPGRADES
WINNIPEG, MANITOBA

SITE LOCATION PLAN



P:\JOBS\17200'S\17250'S\17253 – MCDERMOT AVE OUTFALL UPGRADE\DRAWINGS\WX17253.DWG

CLIENT LOGO: MMM GROUP	CLIENT: MMM GROUP LIMITED	DWN BY: MD	REV. NO.: A
		CHK'D BY: WKW	DATE: NOVEMBER 2013
AMEC Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	amec	DATUM: NAD83	PROJECT NO: WX17253
		PROJECTION: UTM Zone 14 U	FIGURE No. FIGURE 2
		SCALE: AS SHOWN	

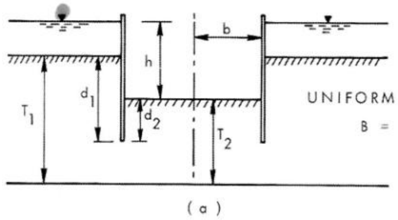
GEOTECHNICAL INVESTIGATION
MCDERMOT AVENUE OUTFALL CHAMBER UPGRADES
WINNIPEG, MANITOBA

RIVERBANK CROSS SECTION ALONG DRAINAGE PIPE

FIGURE 2

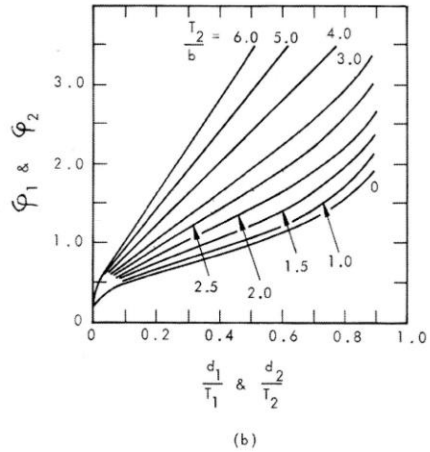
CLIENT LOGO	CLIENT	DWN BY: MD	GEOTECHNICAL INVESTIGATION	REV. NO.: A
MMM GROUP	MMM GROUP LIMITED	CHK'D BY: WKW	MCDERMOT AVENUE OUTFALL CHAMBER	DATE: NOVEMBER 2013
AMEC Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	amec	DATUM: NAD83	UPGRADES WINNIPEG, MANITOBA	PROJECT NO: WX17253
		PROJECTION: UTM Zone 14 U		
		SCALE: AS SHOWN	TEST HOLE LOCATION PLAN	FIGURE No. FIGURE 3

P:\JOBS\17200'S\17250'S\17253 – MCDERMOT AVE OUTFALL UPGRADE\DRAWINGS\WX17253.DWG



NOTE:

Where groundwater level is below ground surface T_1 and d_1 are taken from the groundwater level



1) FOR TWO PARALLEL WALLS

$$i_{exit} = \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$

$$Q = \frac{kh}{\varphi_1 + \varphi_2} \quad \text{PER UNIT LENGTH}$$

φ_1 AND φ_2 FROM FIG. (b)

2) FOR A CIRCULAR EXCAVATION

$$i_{exit} = 1.3\,\frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$

$$Q = 0.8\,\frac{kh}{\varphi_1 + \varphi_2}\;2\pi R$$

3) FOR A SQUARE EXCAVATION

$$i_{exit} = 1.3\,\frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2} \quad \text{(MIDDLE SECTION OF THE SIDES)}$$

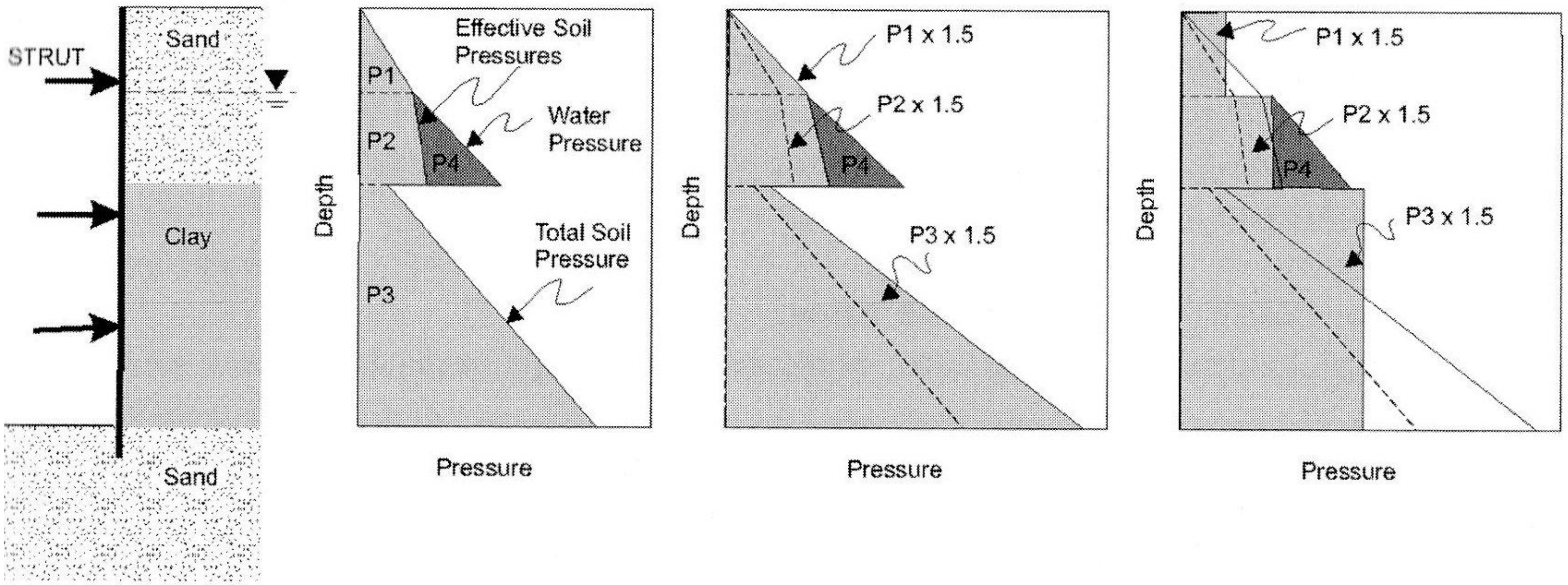
$$i_{exit} = 1.7\,\frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2} \quad \text{(IN THE CORNERS)}$$

$$Q = 0.7\,\frac{kh}{\varphi_1 + \varphi_2}\;4B$$

FIGURE 22.2 *Penetration of sheeting and exit gradient for isotropic sand*

Notes: Figure obtained from CFEM, 4th edition, page 343

amec *Environment & Infrastructure* *A Division of AMEC Americas Limited*		**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**			**ASSESSMENT OF SEEPAGE EXIT GRADIENT** **WINNIPEG, MANITOBA**
Drawn by: WKW	Scale: NTS	Date: NOV 2013	Proj. No: WX17253	Figure: 4	

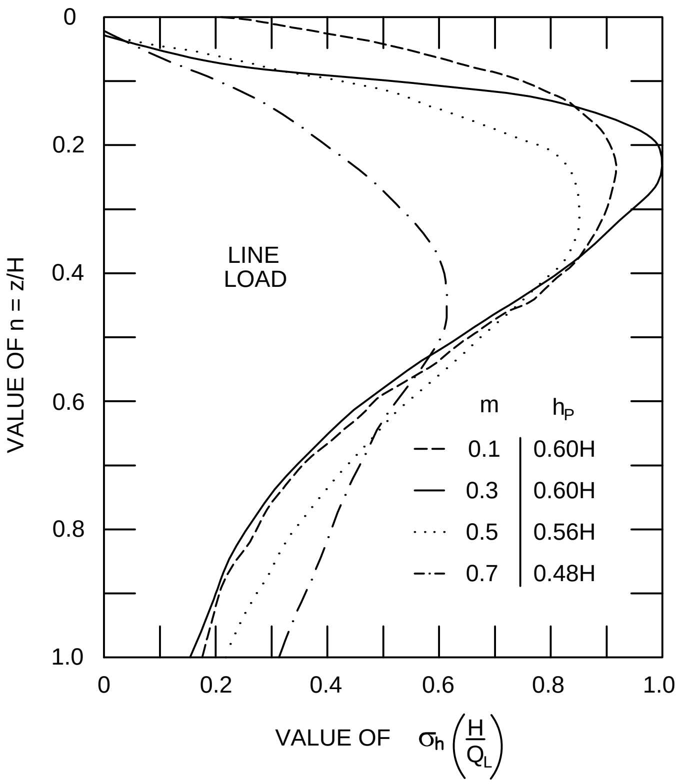


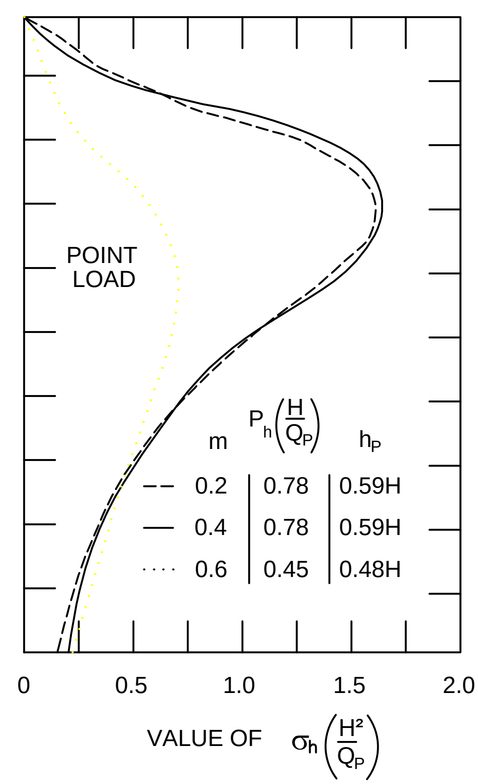
d) method for estimating apparent earth pressures for layered soils

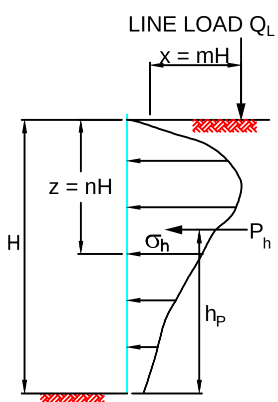
FIGURE 26.8 *Apparent earth pressure distributions*

Notes: Figure obtained from CFEM, 4th edition, page 409

amec *Environment & Infrastructure* *A Division of AMEC Americas Limited*		**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**		**APPARENT EARTH PRESSURE DISTRIBUTIONS FOR BRACED SHORING WALLS** **WINNIPEG, MANITOBA**
Drawn by: WKW	Scale: As Shown	Date: Nov 2013	Proj. No: WX17253	Figure: 5







FOR m ≤ 0.4:

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

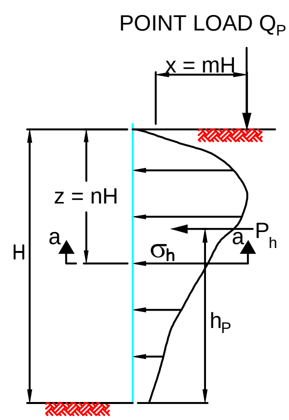
$P_h = 0.55\ Q_L$

FOR m > 0.4:

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

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

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



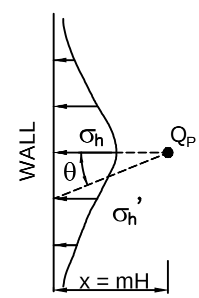
FOR m ≤ 0.4:

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

FOR m > 0.4:

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

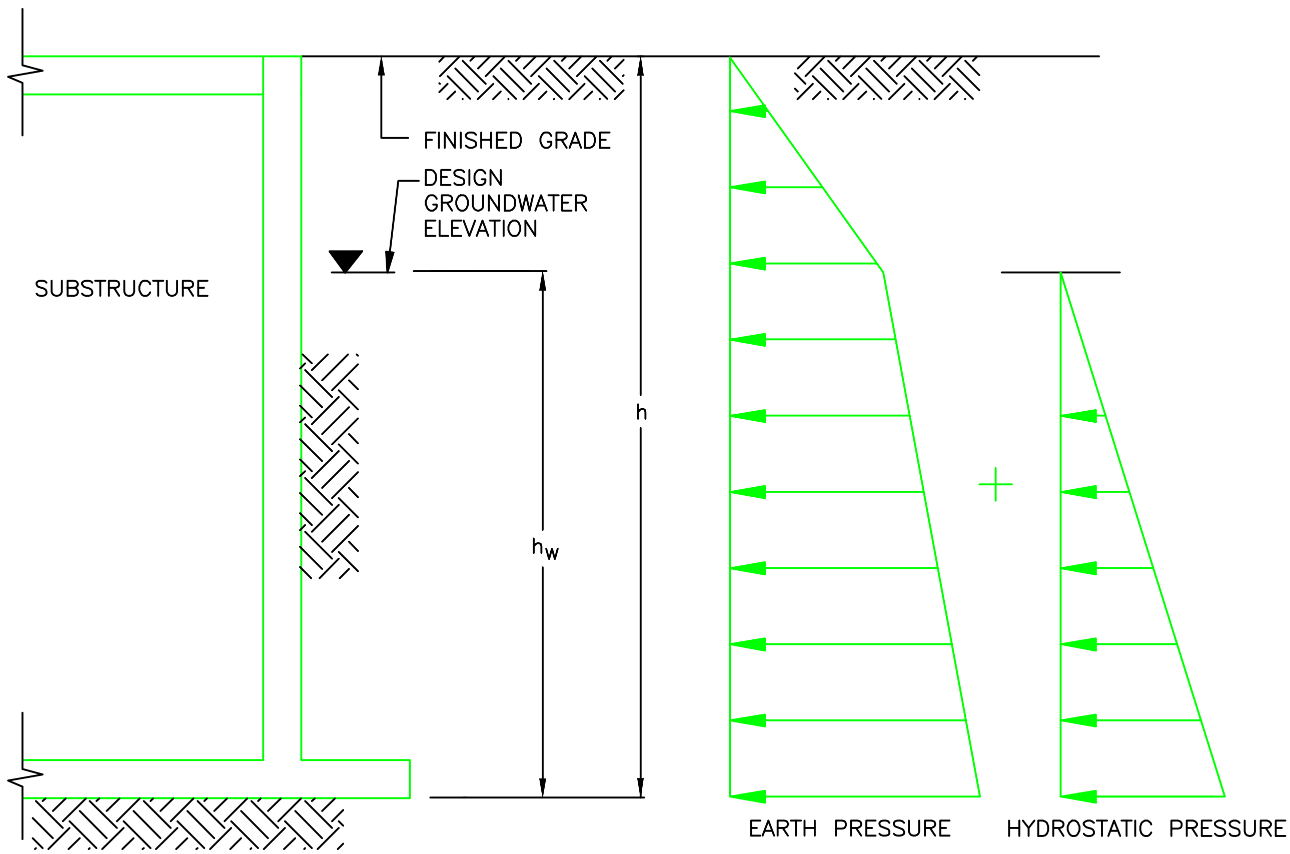
$$\sigma_h' = \sigma_h \cos^2 (1.1\theta)$$



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

MMM GROUP	DWN BY: MD CHK'D BY: WKW	GEOTECHNICAL INVESTIGATION WOODHAVEN OUTFALL UPGRADES 2776-2782 ASSINIBOINE AVENUE WINNIPEG, MANITOBA	DATE: NOVEMBER 2013 PROJECT No.: WX16864
AMEC Environment & Infrastructure 440 Dovercourt Drive, Winnipeg, Manitoba, R3Y 1N4 Office: (204) 488-2997, Fax: (204) 489-8261	DATUM: - PROJECTION: - SCALE: NOT TO SCALE	LATERAL PRESSURES DUE TO SURCHARGE POINT AND LINE LOADS	REV. No.: - FIGURE No.: FIGURE 6

P:\Jobs\16600's\16670's\16679 – Penn-co – Dryden\DRAWING\BRACED SHORING IN TRENCHED EXCAVATION.dwg



RECOMMENDED PRESSURE DISTRIBUTION

$$P_h = K\gamma(h-h_w) + K\gamma' h_w + \gamma_w h_w$$

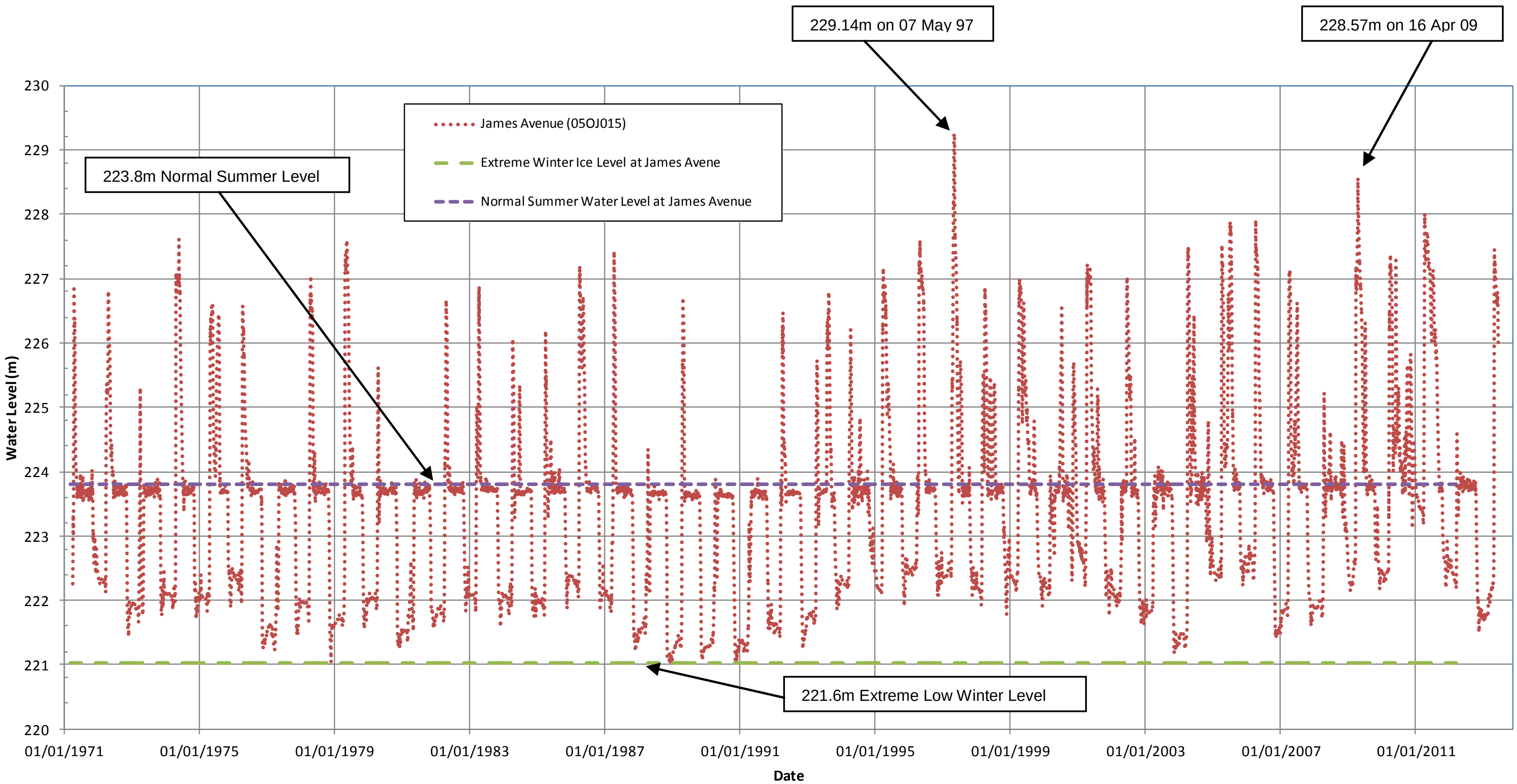
WHERE:
- P_h = LATERAL EARTH PRESSURE (kPa)
- γ = UNIT WEIGHT OF SOIL (SEE TEXT OF REPORT)
- γ' = BUOYANT UNIT WEIGHT OF SOIL (SEE TEXT OF REPORT)
- γ_w = UNIT WEIGHT OF WATER (USE γ_w=9.8 kN/m^3)
- h = HEIGHT OF WALL FROM FINISHED GRADE TO BASE OF WALL (m)
- h_w = HEIGHT OF GROUNDWATER TABLE ABOVE BASE OF WALL (m)
- K = LATERAL EARTH PRESSURE COEFFICIENT (K_o – SEE REPORT TEXT)

NOTES:

1. SEE TEXT OF REPORT FOR UNIT WEIGHTS AND LATERAL EARTH PRESSURE COEFFICIENTS.
2. A HYDROSTATIC COMPONENT NEED ONLY BE INCLUDED WHERE A PERCHED GROUND WATER TABLE WERE TO OCCUR WITHIN THE LEVEL OF SUB–STRUCTURE IN COMBINATION WITH NO SUB–DRAINAGE.

CLIENT: **MMM GROUP**	DWN BY: MD	PROJECT: GEOTECHNICAL INVESTIGATION WOODHAVEN OUTFALL UPGRADES 2776-2782 ASSINIBOINE AVENUE WINNIPEG, MANITOBA	DATE: NOVEMBER 2013
	CHK'D BY: WKW		PROJECT No.: WX16864
amec 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	DATUM: -		REV. No.: A
	PROJECTION: -	TITLE: LATERAL EARTH PRESSURES ON PERMANENT GATE CHAMBER WALLS	FIGURE No.: FIGURE 7
	SCALE: NOT TO SCALE		

P:\Jobs\16300's\16340's\16347 UofM Sifton Road Outfall\drawings\WX16347 – FIG 4.DWG



amec *Environment & Infrastructure* *A Division of AMEC Americas Limited*	**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**	**HISTORICAL WATER ELEVATIONS IN RED RIVER NEAR JAMES AVENUE MONITORING STATION** **WINNIPEG, MANITOBA**

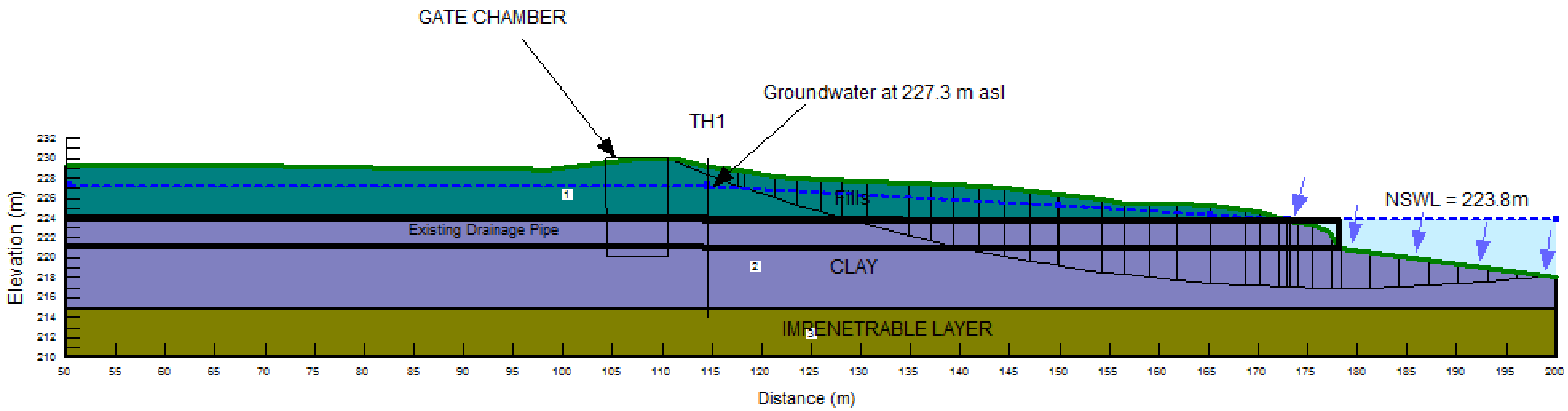
Drawn by: WKW	Scale: As Shown	Date: NOV 2013	Proj. No: WX17253	Figure: 8

Name: Fills
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 1 kPa
Phi: 17 °
Phi-B: 0 °
Piezometric Line: 1

Name: Clay (CI)
Model: Mohr-Coulomb
Unit Weight: 18.5 kN/m³
Cohesion: 1 kPa
Phi: 27 °
Phi-B: 0 °
Piezometric Line: 1

Name: Bedrock
Model: Bedrock (Impenetrable)
Piezometric Line: 1

FS = 2.91



Name: WX17253 - Normal Summer.gsz
Method: Morgenstern-Price

amec Environment & Infrastructure A Division of AMEC Americas Limited	**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**	**RIVERBANK STABILITY ASSESSMENT NORMAL SUMMER DESIGN CONDITIONS** **WINNIPEG, MANITOBA**

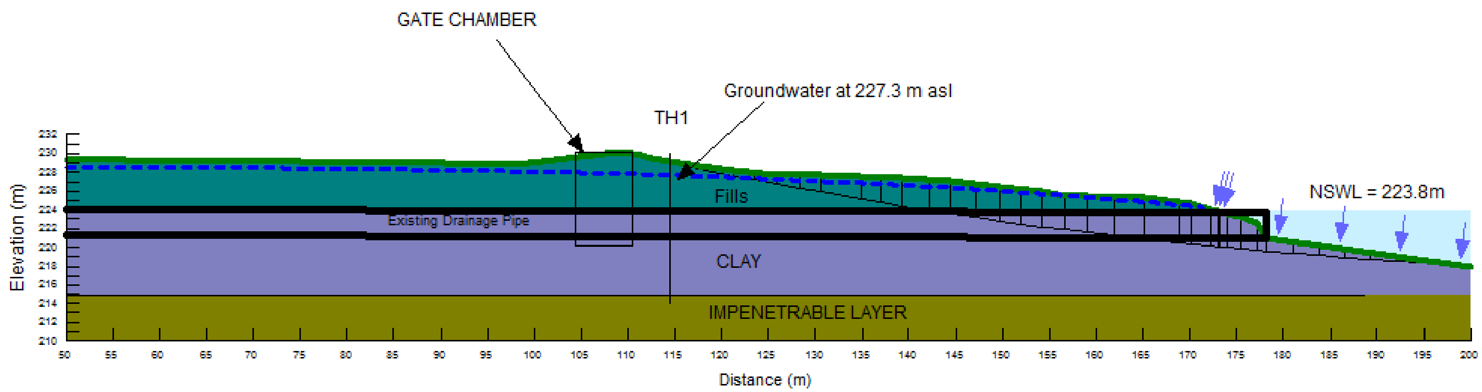
Drawn by: WKW	Scale: As Shown	Date: Nov 2013	Proj. No: WX17253	Figure: 9

Name: Fills
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 1 kPa
Phi: 17 °
Phi-B: 0 °

Name: Clay (CI)
Model: Mohr-Coulomb
Unit Weight: 18.5 kN/m³
Cohesion: 1 kPa
Phi: 27 °
Phi-B: 0 °

Name: Bedrock
Model: Bedrock (Impenetrable)

FS = 2.63



Name: WX17253 - Spring Rapid Draw.gsz
Method: Morgenstern-Price

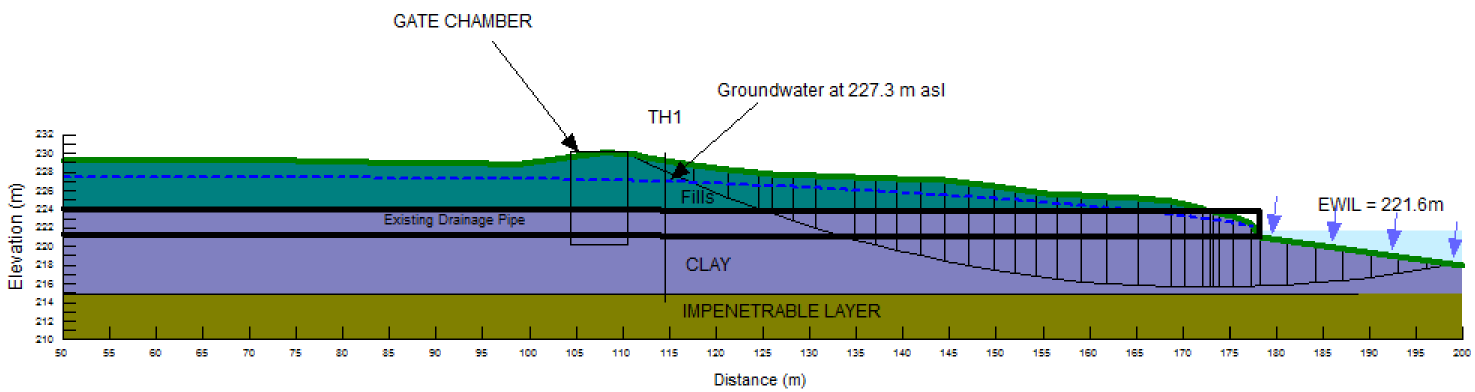
amec *Environment & Infrastructure A Division of AMEC Americas Limited*	**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**			**RIVERBANK STABILITY ASSESSMENT SPRING DRAWDOWN EXTREME DESIGN CONDITIONS** **WINNIPEG, MANITOBA**
Drawn by: WKW	Scale: As Shown	Date: Nov 2013	Proj. No: WX17253	Figure: 10

Name: Fills
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion: 1 kPa
Phi: 17 °
Phi-B: 0 °

Name: Clay (CI)
Model: Mohr-Coulomb
Unit Weight: 18.5 kN/m³
Cohesion: 1 kPa
Phi: 27 °
Phi-B: 0 °

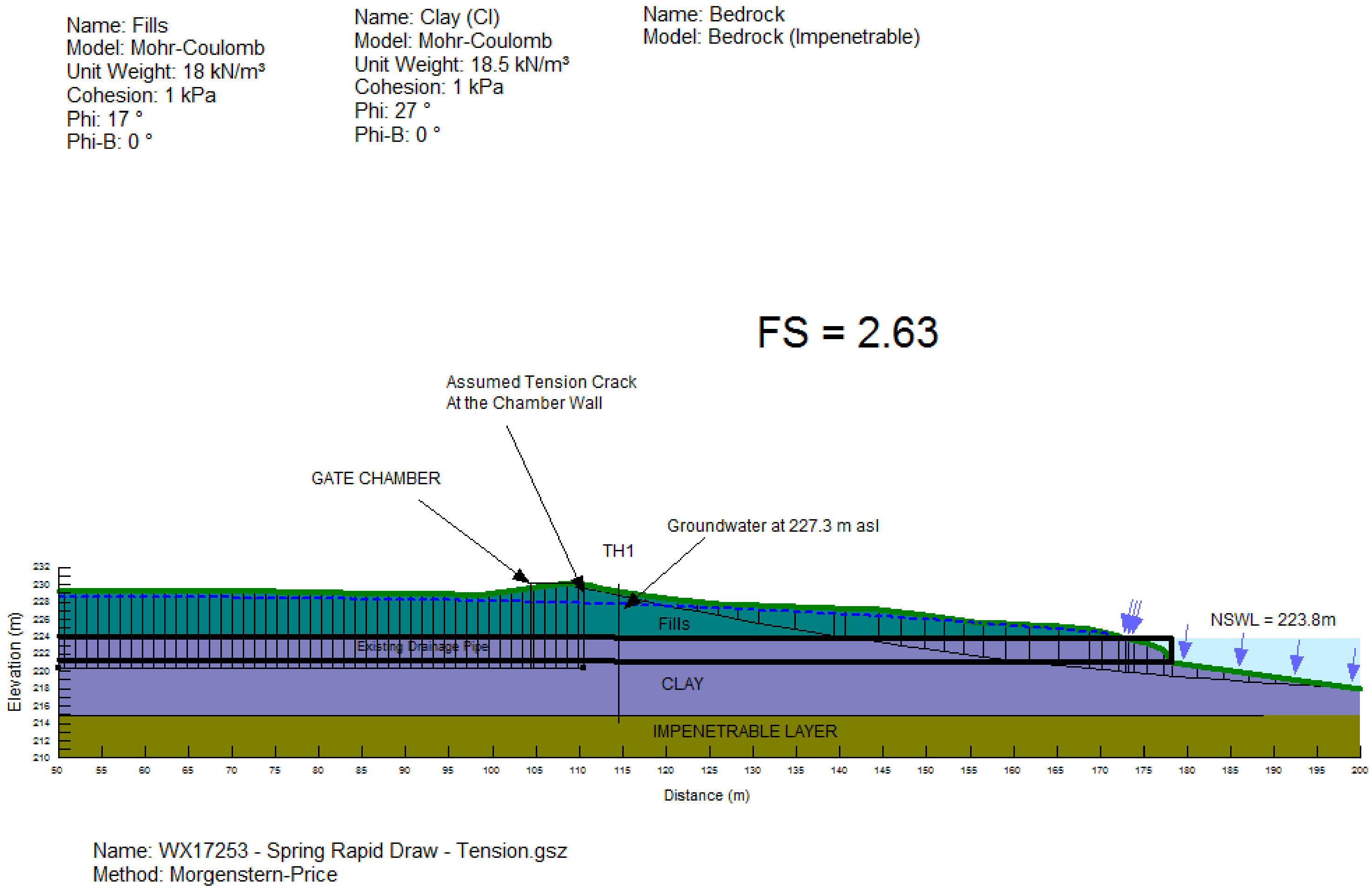
Name: Bedrock
Model: Bedrock (Impenetrable)

FS = 2.58



Name: WX17253 - Fall Draw.gsz
Method: Morgenstern-Price

amec *Environment & Infrastructure* *A Division of AMEC Americas Limited*	**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**			**RIVERBANK STABILITY ASSESSMENT FALL DRAWDOWN EXTREME DESIGN CONDITIONS** **WINNIPEG, MANITOBA**
Drawn by: WKW	Scale: As Shown	Date: Nov 2013	Proj. No: WX17253	Figure: 11



Environment & Infrastructure *A Division of AMEC Americas Limited*	**PROPOSED OUTFALL UPDAGRES AT MCDERMOT AVENUE** **MMM GROUP INC.**	**RIVERBANK STABILITY ASSESSMENT IN PLACE GATE CHAMBER** **WINNIPEG, MANITOBA**

Drawn by: WKW | Scale: As Shown | Date: Nov 2013 | Proj. No: WX17253 | Figure: 12