APPENDIX A

GEOTECHNICAL INVESTIGATION MEMORANDUM

GEOTECHNICAL INVESTIGATION
RUBY AND AUBREY STREET OUTFALL CHAMBERS UPGRADE
WINNIPEG, MANITOBA

Submitted to:
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Attention: Mr. Grantley King, P. Eng.

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1.0 INTRODUCTION

As authorized by Mr. Grantley King of the MMM Group Limited, Amec Foster Wheeler Environment and Infrastructure, a Division of Amec Foster Wheeler Americas Limited (Amec Foster Wheeler), has completed a geotechnical investigation for the proposed Outfall Chamber upgrades at 980 Palmerston (Ruby Outfall) and 1016 Palmerton Avenue (Aubrey Outfall) in Winnipeg, Manitoba.

The scope of work for the geotechnical investigation was conducted in accordance with Amec Foster Wheeler proposal number WPG2016.094, dated 29 February 2016. The purpose of the geotechnical investigation was to investigate the subsurface conditions at the sites 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 in accordance with City of Winnipeg bylaws pertaining to development in proximity to riverbanks.

The following report summarizes the field and laboratory testing programs, describes the subsurface conditions encountered at the test hole locations, and presents geotechnical engineering recommendations for design and construction of the gate chambers at the 980 Palmerston and 1016 Palmerton Avenue.

2.0 SITE AND PROJECT DESCRIPTION

2.1 Project Description

Based on the information provided, it is understood that the Ruby outfall was constructed in the 1960’s and the Aubrey Outfall was constructed in the 1970’s. It is further understood that the outfall chambers are in poor condition and are difficult to operate; and therefore vulnerable to water backs up and reduced capacity during high river elevations. As such, new gate chamber structures are proposed at each location.

The chambers will be constructed to depths ranging from 11 to 12 m from the existing grade. Each new chamber will include a positive gate with electric actuator and a flap gate. A permanently submersible pump will be installed on the upstream side of the chamber to provide greater operational control of the outfall system.

In addition, the project includes the removal of the existing positive gate, 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.

2.2 Proposed Outfall Chambers

Based on the understanding of the project, it is understood that each proposed new gate chamber will have a footing with dimensions of approximately 0.6 m thick and 7.0 m long x 4.2 to 5.3 m wide in plan. The proposed footing will be situated beneath the existing 2.7 to 2.8 m diameter concrete drainage pipe at approximate geodetic elevation 220.2 m.

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

Table 1: Summary of Ruby & Aubrey Outfall Gate Chambers Loading Conditions

Loading Conditions	Unfactored Loads (kN)
Empty Chamber	6530
Chamber Filled with Water	8810

The maximum bearing pressure for both chambers filled with water is approximately 210 kPa.

2.3 Site Description

Amec Foster Wheeler visited the site on 04 May 2016 to perform a site reconnaissance to assess the existing condition of the riverbank for any signs of ground instability that could impact the proposed outfall chambers at the Ruby and Aubrey Sites. Both public and private underground utilities were also located at the time of the site visit. Detailed site descriptions at each proposed outfall chamber location are presented in the following subsections.

2.3.1 Ruby Gate Chamber Site

The Ruby Site is located about 80 m north of the inside bend of the Assiniboine River. The proposed new outfall gate chamber at the Ruby Site will be located southwest of the intersection of Ruby Street and Palmerston Avenue and will be situated in a grass covered island at the northwest corner of the paved parking lot of the Robert A. Steen Community Centre. The approximate location of the proposed gate chamber is presented in Figures 1 and 2. The parking lot is bounded by a wooden fence. The parking lot was relatively flat and gently sloped towards the perimeter as well as to a manhole in the centre of the parking lot. Residential houses were present immediately beyond the western edge of the parking lot. Further south of the parking lot and beyond the wooden fence, was a grass covered slope measuring approximately 4 to 5Horizontal H:1Vertical (5H:1V) over a sloped distance of about 15 m with a chain-linked fence located at the bottom of this slope, approximately at the south property line of the community centre. The area between the chain-linked fence and the Assiniboine River is tree covered, about 8 to 13 m wide and slopes gently down towards the river at gradients of about 5 to 10%. The immediate bank of the Assiniboine River channel, was about 1 to 2 m high above the water level and was about 2H:1V. River elevation was surveyed at 224.808 m on 10 May 2016 by MMM Group.

2.3.2 Aubrey Gate Chamber Site

The proposed gate chamber at the Aubrey Site will be located about 150 m west of the Ruby Site and about 40 m north of the inside bend of the Assiniboine River. The new chamber will be constructed in a relatively flat grass covered opening southwest of the intersection of Aubrey

Street and Palmerston Avenue. The Site is bounded by trees at the western edge of the property, Palmerston Avenue to the north and a City of Winnipeg control station to the east. An existing outfall gate chamber with a footprint of about 3.6 m x 4.3 m was situated about 19 m south of the concrete curb of Palmerston Avenue. About 15 m south of the existing gate chamber was a lookout seating area. The area between the existing gate chamber and the river lookout area was sloped gently at approximately 4 to 6% gradient towards the Assiniboine River. Further south of the lookout area, the slope became about 3 to 4H:1V steeper bank slope over a 14 m distance, followed by a 9 m wide, densely treed, bench adjacent to the Assiniboine River channel. The immediate bank of the Assiniboine River channel, was about 1 to 2 m high above the water level and was about 1H:2V. River elevation was surveyed at 224.816 m on 10 May 2016 by MMM Group.

3.0 FIELD AND LABORATORY PROGRAMS

3.1 Field Investigation

Prior to initiating drilling, Amec Foster Wheeler 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 Foster Wheeler also retained the services of a private utility locator to identify the locations of utility lines in the work areas at both the Ruby and Aubrey Sites.

Amec Foster Wheeler supervised the drilling of two test hole (TH01 and TH02) on a full time basis at the approximate location illustrated in Figures 1 through 3. The test hole TH01 was drilled at the Ruby Site on 12 May 2016 and the test hole TH02 was drilled at the Aubrey Site on 13 May 2016.

The test hole was advanced using a track mounted Acker Renegade drill rig equipped with 125 mm diameter solid stem augers, owned and operated by Maple Leaf Drilling Ltd. (Manitoba) of Winnipeg, Manitoba. Both test holes TH01 and TH02 were drilled to practical auger refusal in the glacial silt till at about 16.9 m (TH01) and 17.2 m (TH02) below the existing ground surface.

During drilling, Amec Foster Wheeler 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 on 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 Foster Wheeler's Winnipeg laboratory. The relative consistency and the undrained shear strength of the cohesive soils encountered at each test hole were evaluated using a hand held Pocket Penetrometer (PP). Relatively undisturbed Shelby tube samples were collected in the alluvial clay for laboratory strength testing. Standard Penetration Tests (SPT) were also conducted within the silt till in conjunction with split spoon sampling.

The sloughing and seepage conditions, as well as the depth to groundwater within the test holes were measured as drilling progressed and immediately after removal of the augers from the test hole. Subsequently, a 25 mm diameter standpipe piezometer was installed in each of the test holes with the slotted section embedded in the glacial till and the bottom of the alluvial clay. The standpipes were backfilled with silica sand, slough / auger cuttings, and bentonite as shown in the test hole logs.

Detailed test hole logs summarizing the sampling, field testing, laboratory test results, and subsurface conditions encountered at the test hole locations are presented in Appendix A, Figures A01 and A02. Actual depths noted on the test hole logs 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 holes, and consisted of the following:

- Moisture content determinations;
- Unconfined compressive strength tests;
- Hydrometer tests (to determine the soil grain size distribution); and
- Atterberg limits (to determine the soil plasticity)

Laboratory test results are summarized on the test hole logs in Appendix A. Test reports for the unconfined compressive strength tests are provided in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy

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

- Various Fills & Organic Clay;
- Alluvial Clays with Interbedded Sand; and
- Glacial Silt Till

<u>Various Fills & Organic Clay</u>

At the Ruby Site, asphalt pavement approximately 90 mm thick was encountered at the ground surface in the parking lot. The asphalt pavement was underlain by a 370 mm thick layer of clay fill.

At the Aubrey Site, organic clay approximately 60 mm thick was present below the grass covered surface. The organic clay was further underlain by a layer of clay fill to about 0.5 m below the existing grade.

The clay fill at both the Ruby and Aubrey Sites was generally characterized as silty, medium to high plastic, stiff and dark greyish brown and contained trace to some amount (up to 20% by soil mass) of sand and gravel. Moisture contents within the clay fill were generally in the range of 22 to 28%.

Alluvial Clays with Interbedded Sand

Alluvial medium to high plastic clay was encountered below the fill and extended to depths of 11.3 m (TH02) to 13.7 m (TH01) below the existing ground surface. The alluvial clay was generally silty, moist, firm to stiff, dark greyish brown and contained some sand increasing to sandy at some depths. Moisture content within the alluvial clay varied from approximately 15% to 55% with an average of 28%.

Frequent sand lenses and layers were present throughout the alluvial clay. The interbedded sand was wet, poorly graded, fine grained, loose to compact (inferred), greyish brown and contained variable clay and silt contents. At TH01, a 1.5 m thick sand layer was encountered at 8.6 m below grade. The sand had a moisture content of 19%.

Grain size distribution of the alluvial clays was assessed by conducting hydrometer tests. The plasticity of the alluvial clays was also tested by performing Atterberg Limits. The grain size and plasticity results of the alluvial clays are presented in Table 2.

Table 2: Grain Size Distribution & Atterberg Limits of Alluvial Clays

No	Location	ID	Plastic Limit (%)	Liquid Limits (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	Ruby	TH01-Sample 3 @ 3m	13	38	0	23	47	30
2		TH01-Sample 15 @ 7.6m	15	51	-	-	-	-
3	Aubrey	TH02-Sample 14 @ 7.9m	50	16	-	-	-	-
4		TH02-Sample 17 @ 9.5m	50	17	0	6	60	34

The unconfined compressive strength of the alluvial clays was tested for both Sites and the results are presented in Table 3.

Table 3: Unconfined Compressive Test Results of Alluvial Clays

No	Location	ID	Maximum Compression Test (kPa)	Strain (%)	Wet Density (kg/m^3)	Dry Density (kg/m^3)	Moisture (%)
1	Ruby	TH01-Sample 7 @ 3m	270.6	8.7	2083	1792	16
2		TH01-Sample 15 @ 7.6m	97.7	5.8	1870	1404	33
3	Aubrey	TH02-Sample 14 @ 7.9m	101	9.9	1870	1404	33
4		TH02-Sample 20 @ 9.5m	57.2	6.3	2002	1502	33

Glacial Silt Till

Glacial silt till was encountered below the alluvial clay and extended to the depth explored at 16.9 m below grade in TH01, and 17.2 m in TH02. The glacial silt till was low plastic, very moist to wet, compact, light greyish brown and contained some sand and gravel and various amounts of clay.

A detailed description of the soil profile encountered at the test hole can be found on the test hole logs in Appendix A , Figures A01 and A02.

4.2 Groundwater and Seepage Conditions

Seepage and sloughing conditions were noted during drilling in the test holes. The depth to slough and accumulated water level within the test holes were measured within about 10 to 15 minutes after completion of drilling. Water levels were also monitored in the standpipes on 30 May 2016, 18 days after drilling. Details of the groundwater conditions, as well as the observed sloughing, are summarized in Table 4.

Table 4: Test Hole Exploration Depths and Groundwater Conditions

Test Hole	Test Hole Depth (m)	On Completion of Drilling		Groundwater Level (m) Below Grade Monitored on 30 May 2016	Groundwater Elevation (m) Monitored on 30 May 2016	Test Hole Surface Elevation (m) asl
		Depth to Slough (m)	Depth to Groundwater (m)			
TH01	16.9	14.6	6.4	5.7	226.38	232.08
TH02	17.2	N/A	8.8	7.2	224.61	231.81
Notes:						

<table>
<tr><th rowspan="2">Test Hole</th><th rowspan="2">Test Hole Depth (m)</th><th colspan="2">On Completion of Drilling</th><th rowspan="2">Groundwater Level (m) Below Grade Monitored on 30 May 2016</th><th rowspan="2">Groundwater Elevation (m) Monitored on 30 May 2016</th><th rowspan="2">Test Hole Surface Elevation (m) asl</th></tr>
<tr><th>Depth to Slough (m)</th><th>Depth to Groundwater (m)</th></tr>
<tr><td colspan="7">- 25mm diameter standpipe was installed to 14.6m below grade in TH01, with screen section from 8.5 to 14.6m in the alluvial clay and till.
- 25mm diameter standpipe was installed to 17.0m below grade in TH02, with screen section from 10.9 to 17.0m in the alluvial clay and till.</td></tr>
</table>

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 greater 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 comprise a cast-in-place concrete footing bearing at elevation of 220.2 m at both of the gate chambers, which is about 11.6 m and 11.9 m below the existing grade at Aubrey and Ruby Sites, respectively. Dependent on the underlying porewater pressure in the glacial till at the time of the excavation, the risk of basal instability and excavation difficulty is considered to be significant for the proposed works to be completed at both the Ruby and Aubrey Sites.
The basal instability and excavation challenges will be affected by the following factors:

- Permeable water bearing soils throughout the excavation depths may lead to sloughing, and seepage as construction proceeds;
- High water pressures below the chamber excavations may lead to piping and heaving of the excavation bases;
- Permeable soils near or just below the base of the excavations may lead to significant seepage, softening and piping at the excavation bases;
- Seepage and softening at the excavation base may further lead to difficulty in establishing a suitable bearing surface for foundation construction; and
- Poor shear strength of the clay at the base of the excavation may lead to failure of the excavation.

The risk will be further influenced by the construction methods, timing and sequencing and would depend on time of year, as aquifer elevations are known to vary significantly on a seasonal basis.

Based on the design embedment depth (i.e. about 12 m below grade) and expected light foundation loads, it is expected that the gate chamber foundation could be designed on the basis of maintaining a net bearing pressure near zero (i.e. weight of structure is approximately the same as the weight of the soil being removed).

On this basis, the following sections provide discussion and recommendations as they pertain to design and construction of the proposed gate chamber, specifically: basal stability and shoring depth considerations; applicable shoring types; lateral earth pressures on chamber walls; temporary construction dewatering requirements; ultimate and serviceability foundation bearing pressures; construction recommendations; and foundation concrete type.

5.2 Excavation Stability

5.2.1 Shoring

Based on the depth of the gate chambers, the soil conditions encountered and the proximity of surrounding existing structures (i.e. buildings, roadways, etc), support will be required to maintain excavation stability for construction of the gate chambers. Currently, it is envisaged that suitable excavation support systems would consist of one of the following braced systems:

- Soldier piles with timber lagging; or
- Sheet piled walls.

Excavations that are extended to a depth of about 12 m below grade will be subject to groundwater issues given that the elevation of the groundwater table was determined to be about 226.4 m (i.e. 6.2 m above the chamber foundation) and 224.6 m (i.e. 4.4 m above the chamber foundation) at the Ruby and Aubrey Site, respectively when monitored on 31 May 2016. Furthermore, the presence of sand lenses / layers within the alluvial deposits suggest that the water levels will likely be heavily tied to the river level; and therefore significantly higher water levels could occur during river flood stages. Therefore, seepage and sloughing from the side walls and seepage the base of the excavation should be anticipated, making the use of internal and/ or external dewatering systems to be a likely requirement for the project.

Generally, there are three base failure modes that must be evaluated specific to the design of a supported excavation as follows:

1. Shear failure;
2. Piping; and
3. Heave

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

5.2.2 Base Stability Against Shear Failure

The stability of the excavation base against shear failure must be evaluated to confirm a safe excavation base condition. This failure mechanism occurs as a result of inadequate resistance of the loads imposed by the differences in grades between the inside and outside of the excavation. According to Canadian Foundation Engineer Manual (CFEM), if the FS against base 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 designing a shoring system for stability against base shear failure, a FS greater than 2.0 should generally be targeted. If FS is less than 2.0, substantial deformation may occur.

Ruby Site:

Based the proposed excavation dimensions of about 4 m wide, 7 m long and about 11.9 m deep at the Ruby gate chamber, the FS against shear failure is determined to be considerably less than 1.5. As a result, the depth of the penetration of the shoring should be extended into the till to protect against potential base shear. For a soldier pile shoring system with timber lagging, the soldier piles would need to be closely spaced to prevent any instability or failure of soil from the exterior to interior of the excavation through and between the supports. A suitable shoring supporting system could consist of a continuous sheet piles, tangent or secant pile wall system.

Aubrey Site:

The proposed gate chamber of the Aubrey Site is expected to create an excavation of about 5.4 m wide, 7 m long and about 11.6 m deep. For this Site, the base of the excavation would be founded within the till and as such, the FS against base shear failure is greater than 2.0. Therefore, the design requirement for this mode of failure is met without extending the shoring below the base of the excavation. In this case, a soldier pile and timber lagging shoring system may be possible at the Aubrey Site.

5.2.3 Base Stability against Piping Failure

Given the groundwater elevations and excavation depths determined above for each of the Sites, pressure heads ranging from 4.4 m to 6.2 m above the base of the excavations are expected and as a result significant seepage should be anticipated. The seepage rate will depend on the actual soil conditions at the excavation, most importantly 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 these 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 this basal stability condition may be determined by following the method provided in Section 22.3.2.1 of the 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 0.9
- FS = Factor of Safety; taken as 2.0
- C = Constant; taken as 1.3 for circular and middle section of the square sides; and 1.7 for the corners of 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; b = 3.5 m for both the Ruby & Aubrey Sites)
- 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 214 m.
- T_2 = Depth from the excavation base to an impervious layer at depth. Assuming the impervious layer as the bedrock layer at approximately 214 m.

It should be noted that the value of ‘h’ provided above is determined based on groundwater monitoring results determined from monitoring conducted on 31 May 2016. Increases in groundwater elevation can occur due to heavy rains, rises in the nearby river level, rises in the underlying bedrock aquifer level, which may be connected to the glacial till layer or also due to nearby construction activities. Amec Foster Wheeler recommends that groundwater conditions be monitored prior to and during construction to verify the basal stability of the excavation and the design shoring penetration depth. As well, the construction period should be reviewed so that the likelihood of higher water levels during construction can be determined and the risk reduced accordingly.

The above method for exit gradient assessment will allow determination of whether basal stability will be of concern for a given groundwater condition and/or the depth of shoring penetration required beneath the excavation depth. Dewatering will be needed if the required shoring length is not achievable. Based on the current groundwater conditions encountered on Site, Amec Foster Wheeler has determined that even where the shoring is installed into the glacial silt till, there is a potential risk for basal instability by piping to occur. As a result, groundwater control such as dewatering of the existing groundwater level below the base of the excavation is needed to improve the basal stability. The details of construction dewatering are discussed in Section 5.2.8.

5.2.4 Soils Heave at the Excavation Base

In addition to the exit gradient assessment, the potential for soil heave to occur at the base of the excavation should also be checked. This failure mode can occur in cases where an impervious layer (i.e. clay) is present at the excavation base, overlying a pervious layer (i.e. sand or till) which contains groundwater under pressure. For this case, the porewater pressure at the top of the pervious layer should not exceed 70% of the total vertical stress at the top of the pervious layer. If this condition cannot be satisfied, then a greater sheet pile penetration depth and dewatering at greater 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 5, times the total thickness of the soils.

In order to meet the following condition to limit base heaving, it is determined that the groundwater tables at both the Ruby and Aubrey Sites need to be reduced to the base of the excavation at 220.2 m. It is should be noted that in order to protect the bearing surface of the mat foundation for the gate chamber from disturbance, the groundwater should be further dewatered to a minimum 1 m below the bearing surface. Details of dewatering are presented in Section 5.2.8.

5.2.5 Summary of the Shoring System

According to the aforementioned discussions, an appropriate shoring system on each site can be summarized as follows:

Ruby Site:

- The approximately 2 m thick clay soil beneath the bearing surface of the proposed chamber at 220.2 m is weak with undrained shear strength estimated of 30 kPa and as a result the excavation is prone to base shear failure. Therefore, a shoring supporting system would need to penetrate below the excavation elevation and preferably into the underlying till; and
- Dewatering will be required at this Site to prevent base failure due to heaving and piping;
- Therefore, a braced sheet pile wall shoring system driven into the till is recommended for this Site.
- A braced soldier pile with timber lagging shoring system is not preferred due to the potential for seepage, sloughing and base shear failure to occur between soldier piles beneath the base of the excavation;

Aubrey Site:

- The proposed chamber will be bearing directly on the glacial till that has higher soil strength than clay. Therefore, the base shear mode is not critical and a shoring supporting system does not necessarily need to penetrate below the excavation elevation; and
- Dewatering will be required at this Site to prevent base failure due to heaving and piping.

- Therefore, either a braced soldier pile with timber lagging system or a braced sheet pile system is considered to be feasible; however, a braced sheet pile wall shoring system is recommended for the Site to control seepage and sloughing from the excavated walls.

5.2.6 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 given 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 26.8b and the following soil parameters. The Figure 26.8b in the CFEM is reproduced in Figure 5 of the report.

Table 5: Lateral Earth Pressure Coefficients on the Gate Chamber Walls

Soils Parameter	Alluvial Medium to High Plastic Clay	Glacial Silt Till
ϕ', Internal Friction Angle	23	32
γ_t, total unit weight (kN/m^3)	19	20
γ' , submerged unit weight (kN/m^3)	9.2	10.2
"At-rest" Earth Pressure Coefficient, K_o	0.61	0.47
"Active" Earth Pressure Coefficient, K_A	0.44	0.31
"Passive" Earth Pressure Coefficient, K_p	1.72	2.14

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

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

where,

K = Earth Pressure Coefficient
σ_v = Total Vertical Stresses (γ x H_o)
H_o = Embedment Depth of Wall Below Grade (m)

This lateral earth pressures will then be applied to 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 base. 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.

The passive earth pressure coefficients provided in Table 5 include a reduction factor of 1.5 to the friction angle to account for the partial mobilization of passive resistance that is consistent with the small wall displacements expected under operational conditions. Relatively large wall displacements would be necessary to realize full passive resistances. To determine the factored resistance, a resistance factor (Φ) of 0.5 should be applied to the resulting passive earth pressure determined using the parameters provided in Table 5.

Total unit weights of the soils should be used above the water table. A combination of submerged soil unit weights and horizontal pressure resulting from the design static water level should be used below the water table. The design water level 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 lateral earth 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 Foster Wheeler can provide the lateral earth pressures distributions of the proposed shoring system once the details of the excavation and shoring type are finalized.

5.2.7 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.

5.2.8 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 to allow for development of a clean, dry and stable subgrade for construction of foundations. In addition, even where an external dewatering system is implemented, there may be potential of slight water seeping into the excavation. If this occurs, or where redundancy is needed, an internal dewatering system (i.e. pumping water inside of the excavation) should also be implemented. The internal dewatering system should be used to control potential 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 comprise 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 alluvial soils or glacial till. 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 Foster Wheeler during construction to determine the effectiveness of the external and internal dewatering system as well as assess the subgrade bearing surface.

5.2.9 <u>Soldier Piles and Timber Lagging System</u>

It is understand that typically, a solider pile and timber lagging system is preferred by contractors and typically used for gate chamber excavation in the City of Winnipeg. However, as discussed above, given the presence of weak clay below the base of the excavation at the Ruby Site and the high groundwater conditions in the glacial till and alluvial clays at both the Ruby and Aubrey Sites, the use of such a shoring system is not recommended and may lead to basal instability if the proper controls are not put in place. The solider pile and timber lagging system is only considered to be possible, if the following minimum conditions are met:

1. Excavate base of chambers to a maximum of 1 m above the glacial till to reduce the potential for base shear failure;
2. Dewater groundwater adjacent to and below the chamber excavations to a minimum 1 m below the base of the excavation or alternatively conduct excavation at time of year when natural groundwater conditions meet this condition; and
3. Control water seepage through the excavated walls.

If the above minimum conditions are met, it is expected that solider and timber lagging shoring can be utilized. Typically, for shallow excavations a cantilevered system would be used; however, given the depth of the excavation, internal bracing or tie backs will be needed.

Lateral earth pressures that are needed for the solider piles and timber lagging system shoring design can be obtained from Section 5.2.6. It should be noted that the passive earth pressure for the solider piles below the excavation base should be applied to the flange width of the piles

Generally in Winnipeg, H-piles are utilized as soldier piles and are installed in one of two manners:

- Driven to refusal in the glacial silt till or bedrock
- Drilled hole with H-piles concreted in place in the till, such that the flat faces of the H-piles are directed toward the interior of the excavation.

During the pile installation, vibrations created during pile driving may affect the nearby existing structures. The effect of vibration can be reduced by pre-drilling the pile hole (i.e. to about 6 m) as opposed to driving the pile right from the ground surface.

If solider piles are installed in a drilled hole, the contractor should be use a protective steel casing to maintain the pile holes in an open and dry condition. Where seepage cannot be controlled, all concrete will have to be placed using tremie methods. In addition, the glacial silt till commonly contains cobbles and boulders and therefore drilling of the steel soldier piles holes may require the removal of these obstructions.

Following installation of the piles, the soil in front of (i.e. on the interior of the proposed excavation) and immediately between the piles is excavated in a staged manner to ensure stability of the shoring system. During each stage, the timber lagging boards are placed between the pile flanges and bolted as required. At pre-determined depths steel anchors or struts can be installed. To prevent seepage and soil migration through the small gaps between the lagging boards, a non-woven geotextile should be installed behind the wood lagging.

5.2.10 <u>Excavation Staging</u>

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.

5.2.11 <u>Shoring Wall Monitoring</u>

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 will also depend 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 Foster Wheeler 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.

5.2.12 <u>Other Considerations</u>

It should be noted that there are additional issues that should also be considered for a temporary shoring system that 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 are 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 <u>Design Footing Bearing Pressure</u>

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 2.7 (Ruby Site) to 2.9 m (Aubrey Site) diameter concrete drainage pipe. The depth of the pipe at the gate chamber location is estimated to be at approximately 12 m below existing ground surface of the proposed gate chamber location at approximately 220.2 m. In addition, the proposed footing will be 4.2 to 5.4 m wide and 7 m long.

On this basis, the ultimate bearing capacity of the bearing soil at both the Ruby and Aubrey Sites are follows:

Ruby Site

The ultimate bearing capacity of the alluvial clay, which will form the foundation for the concrete footing at the Ruby Site, is 480 kPa. A geotechnical resistance factor of 0.5 should be applied to the ultimate bearing capacity and compared to the factored loads under the limit state design approach. As a result, the proposed footing will have a factored geotechnical resistance of 240 kPa.

According to the information provided to Amec Foster Wheeler, it was understood that the proposed gate chamber foundation will have a maximum unfactored bearing pressure of 210 kPa when water is in the chamber. Total settlement of the footing under a 210 kPa service load is estimated to be between 20 and 40 mm.

Aubrey Site

The ultimate bearing capacity of the glacial silt till, which will form the foundation for the concrete footing at the Aubrey Site is 600 kPa. A geotechnical resistance factor of 0.5 should be applied to the ultimate bearing capacity and compared to the factored loads under the limit state design approach. As a result, the proposed footing will have a factored geotechnical resistance of 300 kPa.

Considered that the details of the Aubrey gate chamber are not available, it is assumed that the design maximum unfactored bearing pressure will also be approximately 210 kPa when water is in the chamber, which is similar to the gate chamber at the Ruby Site. Total settlement of the footing under a 210 kPa service load is estimated to be between 15 and 35 mm.

For both sites, 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). It should be noted that additional settlement could occur where disturbance and/or softening of the bearing surface occurs during construction. The expected settlement should be reviewed and where the amount of settlement is not tolerable, Amec Foster Wheeler can modify the serviceability limit state on request accordingly.

5.3.2 Buoyancy

Based on the anticipated groundwater level at the Sites, the gate chambers will be subject to uplift pressure due to buoyancy. For design purposes, the buoyancy force may be estimated assuming a groundwater table at the design maximum flood level of about 231 m (700 year flood protection level) for this part of the Assiniboine River. The gate chamber should be designed to resist buoyancy in situation for a design flood event. 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 factored 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 12 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 that is 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 would 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 Foster Wheeler can provide lateral earth pressure distributions for highly compacted backfill on 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 should be taken as the design flood level (i.e. 231 m of 700 year flood protection level) for the project on site.

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 5 in Section 5.2.6.

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 between the walls of the chamber and native soils. 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 will impose uplift forces 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 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 12 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 due to frost and uplift due to buoyancy result from different mechanisms and they occur during different seasons. On this basis, these conditions are not additive and should be addressed separately in design.

5.4 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.

5.5 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

[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.

- 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 Foster Wheeler requests the opportunity to review the design drawings and the installation of the gate chambers to confirm that the geotechnical recommendations have been correctly interpreted. Amec Foster Wheeler further requests the opportunity to review the soil and groundwater conditions encountered as excavation proceeds 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 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.

5.6 Riverbank Slope Evaluation

5.6.1 <u>Slope Stability Criteria</u>

The project site is located on an inside bend of the north bank of the Assiniboine River in Winnipeg. Since the site is located within about 100 m of the Assiniboine River, the proposed works will require securing of 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."*

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

5.6.2 Slope Stability Evaluation

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 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 each 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 and composite (i.e. circular slip surface intersecting a hard till layer) slip surfaces were estimated using the entry and exit method and the Morgenstern-Price method with a half sine variation of inter-slice forces.

5.6.3 Topography

Site topography was surveyed by MMM Group on 10 May 2016 along the approximate alignment of the existing drainage pipe and at the surrounding area of the riverbank from the upper slope of the riverbank at Palmerston Avenue to the edge of the Assiniboine River channel. Bathymetric survey was not performed for the project to determine the river channel profile below the water level. The river channel profiles for both the Ruby and Aubrey Sites along the existing pipeline alignment were estimated from the channel profiles extracted from the Hydrologic Engineering Center – River Analysis System (HEC-RAS) that the City has collected. It is understood that the river channel profiles were based on the bathymetry and LiDAR surveys performed in 2011. The locations of the river channel profiles provided by the City are presented in Figure 1. Cross-sections used for stability modelling at the Ruby and Aubrey Sites are illustrated in Figures 2 and 3, respectively.

5.6.4 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 Foster Wheeler. In this regard, the effective shear strength parameters used for the soil strata observed in test holes for both the Ruby and Aubrey Sites were assumed based on commonly used strength parameters for similar observed soils in Manitoba, plus the experience from Amec Foster Wheeler's previous projects.

The simplified soil stratigraphy used for development of the stability models for each site are summarized as follows:

<u>Ruby Site:</u>

- 0 to 0.5 m Asphalt and clay fill;
- 0.5 to 13.7 m Medium to high plastic, silty, alluvial clay with interbedded sand lenses;
- Below 13.7 m Glacial silt till;

Groundwater level in the standpipe was recorded at 5.7 m (about El. 226.4 m) on 31 May 2016.

<u>Aubrey Site:</u>

- 0 to 0.5 m Topsoil and clay fill;
- 0.5 to 11.3 m Medium to high plastic, silty, alluvial clay with interbedded sand lenses;
- Below 11.3 m Glacial silt till;

Groundwater level in the standpipe was recorded at 7.2 m (about El. 224.6 m) on 31 May 2016.

Note that for simplification, the stability models have ignored the presence of any asphalt, topsoil or clay fill.

Visual evidence of previous riverbank movements such as tension cracks, slumps, soil rotation, failure scarps and samples containing slickensided surfaces, etc. was not observed either during the field reconnaissance or within the geotechnical samples recovered from the test holes for this project. Given the lack of evidence of previous slope movement, a distinct shear zone was not included in the model. Further, the use of fully softened or post peak effective shear strength parameters was considered to be appropriate for the alluvial clay soils encountered at this site. These parameters are summarized in Table 6. 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 and takes 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 6: Summary of Average Isotropic Shear Strength Parameters

Parameter	Alluvial Clays	Glacial Silt Till
Unit Weight, γ (kN/m^3)	19	Bedrock[1]
Effective Post Peak Shear Strength Parameters	$\phi' = 23°$ $c' = 3$ 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.

5.6.5 Assiniboine River Levels

The proposed Ruby and Aubrey Sites are situated about 1 km west to the City of Winnipeg's Maryland Bridge river level monitoring station. As a result, the monitoring data was obtained from the City as a reference for the river level conditions at these Sites. As mentioned earlier, the site topographic survey was performed on 10 May 2016. At the time, the river elevations at both the Ruby and Aubrey Sites were surveyed to be at 224.81 m and 224.82 m, respectively. The river elevation at Maryland Bridge monitoring station was recorded to be about 224.73 m on the same day. Therefore, it is concluded that the river elevations between the Sites and the monitoring station at Maryland Bridge are approximately the same. On this basis, the historical river elevation data recorded from the Maryland Bridge monitoring station are considered to be suitably representative of those at the Ruby and Aubrey Sites and have been used for the stability assessments. Approximately 10 years of historical river level data from 2006 to 2016 at the Maryland Bridge monitoring station are illustrated in Figure 8.

As presented in Figure 8, the river levels fluctuate seasonally with high river levels occurring in the spring and low river levels in the winter with typical fluctuations of about 5 m observed annually. Based on this data, the Assiniboine River conditions selected for the stability assessments are estimated to consist the following:

a) Normal Summer River Water Level (NSRWL): 224.0 m;
b) Extreme Winter Ice Level (EWIL): 222.2 m; and
c) Extreme Spring River Water Level (ESRWL): 229.0 m

5.6.6 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. Given that the alluvial riverbank soils contain frequent silt and sand lenses and/or layers, the groundwater conditions in the riverbank are expected to be respond quickly and with a similar magnitude of change as any changes in the river elevations. That is to say that the lag time between changes in river elevations and changes in groundwater elevations is expected to be relatively short.

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 somewhat 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.

Historical groundwater level data at a provincial monitoring well station (i.e. G05MJ043), about 1.5 km east of the site and about 50 m away from the Assiniboine River, was collected as a reference for the groundwater conditions in the bedrock at both the Ruby and Aubrey Sites. This data is presented in Figure 8 together with the river level data. As shown in the plots, the groundwater levels at the bedrock are generally 1 to 2 m lower than the river levels in the spring and summer and about 1 m higher than the river levels in late fall and winter.

As indicated in Section 3.1, 25 mm diameter standpipes were installed such that the slotted section of the standpipes intersected both the alluvial clay and glacial silt till at both the Ruby and Aubrey Sites. Groundwater monitoring data was collected at each of these locations on 31 May 2016 (i.e. 19 days after the standpipes were installed). Groundwater levels of about 226.4 m and 224.6 m were recorded at the Ruby and Aubrey Sites, respectively. The groundwater level recorded at the provincial bedrock monitoring well (i.e. G05MJ043) was recorded at 224.52 m on the same day, which is similar to the groundwater condition at the Aubrey Site. The river level, also recorded on the same day, was 224.7 m and was also approximately the same as the water level in the bedrock monitoring well. As a result, the groundwater elevation at the Aubrey Site appears to be similar to that of both the river and the bedrock observed at the monitoring stations. The groundwater level recorded in the standpipe at the Ruby Site is about 1.9 m higher than recorded at the monitoring well and about 1.7 m higher than the recorded river level. The reason for the apparent higher groundwater conditions at the Ruby Site is unknown. The difference may be related to the position of the standpipes and bedrock monitoring well relative to the river's edge. In this regard, the standpipe at the Ruby Site is about 80 m away from the river's edge; the standpipe at the Aubrey Site is about 45 m away from the river's edge; and the monitoring well is about 50 m away from the river's edge. Therefore, the groundwater conditions at the Aubrey Site and the monitoring well are likely more highly influenced by the Assiniboine River conditions than the conditions at the Ruby Site, which is further away from the edge of the river.

5.6.7 Stability Modeling Approach & Results

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 NSRWL. This condition is based on the interpreted NSRWL together with the existing groundwater monitoring data collected near the proposed location of the gate chamber. The estimated river and groundwater elevations are as follows:

    - River conditions @ NSRWL = 224.0 m;
    - Groundwater level at gate chamber = 226.0 m (i.e. 2 m higher than the NSRWL)

2. Spring Drawdown Extreme Design Conditions – These conditions were selected to reflect the extreme condition that is presumed to occur following drawdown from the ESRWL to the NSRWL. The effect of the drawdown was evaluated using seepage modeling (Seep/W) to estimate the variation in riverbank porewater conditions as the river is drawn down from the ESRWL (i.e. 229.0 m) to the NSRWL (i.e. 224.0 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 NSRWL to the EWIL. The effect of the drawdown was evaluated by using seepage modeling to estimate the variation in riverbank porewater conditions the river is drawn down from NSRWL (i.e. 224.0 m) to the EWIL (i.e. 222.2 m) over a period of one month that is typically observed from the historical river level trend.

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 as they will not be impacted by the proposed gate chamber development.

Slope Stability Results at Ruby Site:

From the above three (3) scenarios, factors of safety (FS) are determined and shown in Table 7. These results all exceed the minimum FS requirements identified in Section 5.6.1 for the extreme and normal conditions at the proposed gate chamber location. The results of these scenarios are presented in Figure 9 to 11.

Table 7: Riverbank Stability Results at Ruby Site

Scenarios	FS
Scenario 1: Normal Summer Design Conditions	3.76
Scenario 2: Spring Drawdown Extreme Design Conditions	3.34
Scenario 3: Fall Drawdown Extreme Design Conditions	3.41

Slope Stability Results at Aubrey Site:

The stability assessment on the Aubrey Site was performed considering that the proposed gate chamber may be situated either 7 m to the north (i.e. Case 1) or 7 m to the south (i.e. Case 2) of the existing chamber under the above three (3) scenarios. Under these conditions, the factors of safety are determined and shown in Table 8.

Table 8: Riverbank Stability Results at Aubrey Site

Scenarios	FS	
	Case 1	Case 2
Scenario 1: Normal Summer Design Conditions	2.15	1.97
Scenario 2: Spring Drawdown Extreme Design Conditions	1.90	1.70
Scenario 3: Fall Drawdown Extreme Design Conditions	2.05	1.85

Notes: Case 1 – New gate chamber to the north of the existing chamber

These results all exceed the minimum FS requirements identified in Section 5.6.1 for the extreme and normal conditions at the proposed gate chamber location. The results of these scenarios are presented in Figures 12 to 14 for Case 1 and Figures 15 to 17 for Case 2.

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 chambers will not:

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

5.6.8 Erosion Protection

Based on site reconnaissance performed, it was observed that little to no riprap protection was present along the riverbanks of the Assiniboine channel at the both Ruby and Aubrey Sites. In general, riverbanks without erosion protection would be vulnerable to bank toe erosion, which may lead to riverbank instability. The project sites at Ruby and Aubrey are situated at the inside bend of the river where the erosion condition would typically not be as critical as for those riverbanks located at the outside bend, which would be subject to continuous flows against the bank and therefore eroding soils away from the riverbank. Riverbanks at the inside bends generally receive fines deposition.

Considering that a hydraulic study has not been performed for this project, neither the river flow volume nor flow velocity are known at the proposed sites. As a result, the erosional condition on site is difficult to assess and therefore it is not possible to determine the erosion protection requirements on site. Unless a hydraulic study is performed to determine the requirement for erosion protection on the both sites, it is recommended that as a good practice, an erosion protection riprap blanket covering a layer of non-woven geotextile be placed around the discharge area at the outfall locations. Amec Foster Wheeler can provide a further detailed design for erosion protection blankets on request.

5.6.9 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 Foster Wheeler should be contacted to evaluate the riverbank stability pertaining to these conditions.

6.0 CLOSURE

The findings and recommendations presented herein for design of the proposed Ruby and Aubrey outfall gate chambers are based on a geotechnical evaluation of the findings in the geotechnical test hole drilled at the sites. 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 Foster Wheeler 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 Foster Wheeler.

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

PROVINCE OF MANITOBA
16 June 2016
W. K.
WONG
Member
23538
REGISTERED PROFESSIONAL ENGINEER

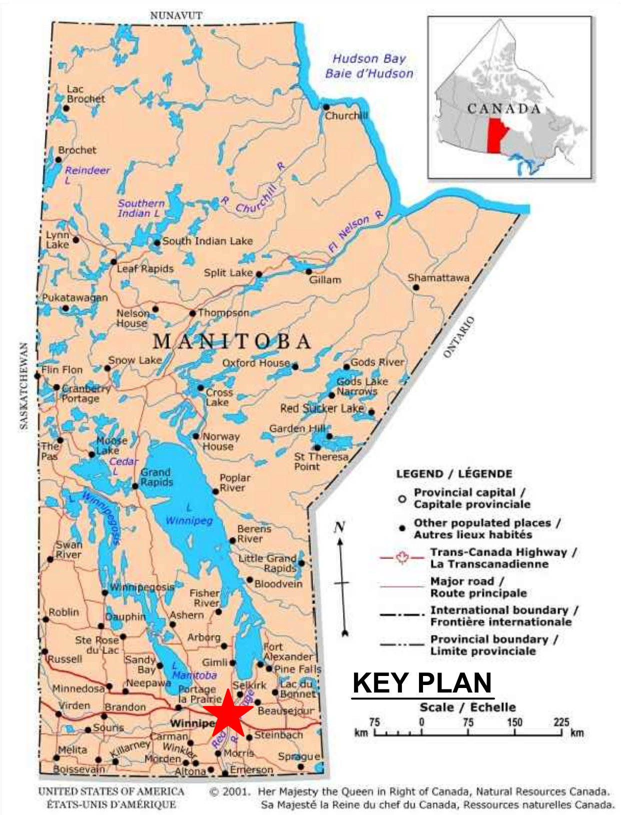
APEGM
Certificate of Authorization
Amec Foster Wheeler Environment & Infrastructure, a Division of Amec Foster Wheeler Americas Limited
No. 6017 **Date:** 16 June 2016

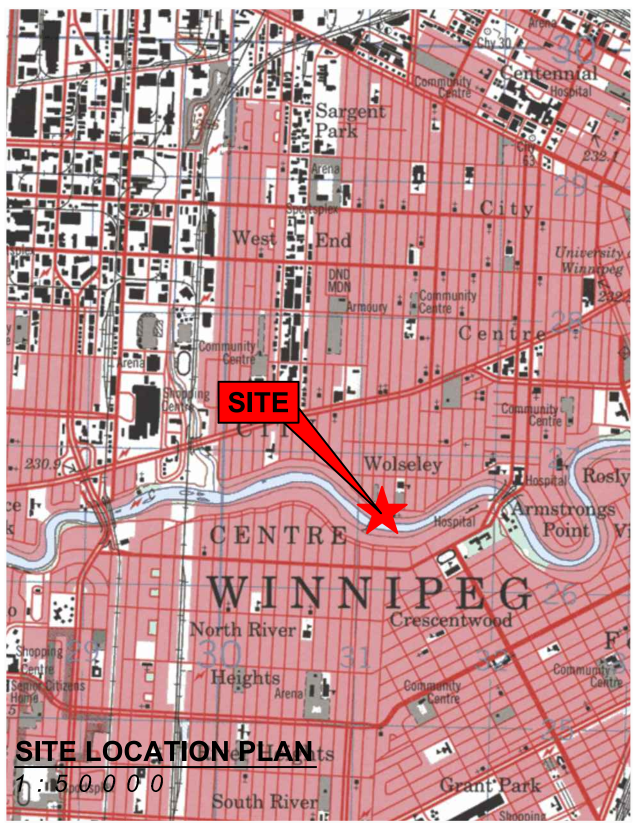
Reviewed by:

Wing-Keat (Wayne) Wong, M.Eng, P. Eng.
Senior Geotechnical Engineer

Brad Wiebe, M.Sc., P.Eng.
Associate Geotechnical Engineer
Manager of Geotechnical Services

FIGURES





- IMAGE FROM MLI WEBSITE

LEGEND:

RUBY SITE TEST HOLE — TH01

AUBREY SITE TEST HOLE — TH02

CITY OF WINNIPEG HEC-RAS CROSS SECTION (2011 BATHYMETRIC & LIDAR SURVEY DATA)

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

CLIENT:

MMM GROUP LIMITED

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:	WKW
DATUM:	NAD83
PROJECTION:	UTM Zone 14 U
SCALE:	AS SHOWN

GEOTECHNICAL INVESTIGATION
RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE
WINNIPEG, MANITOBA

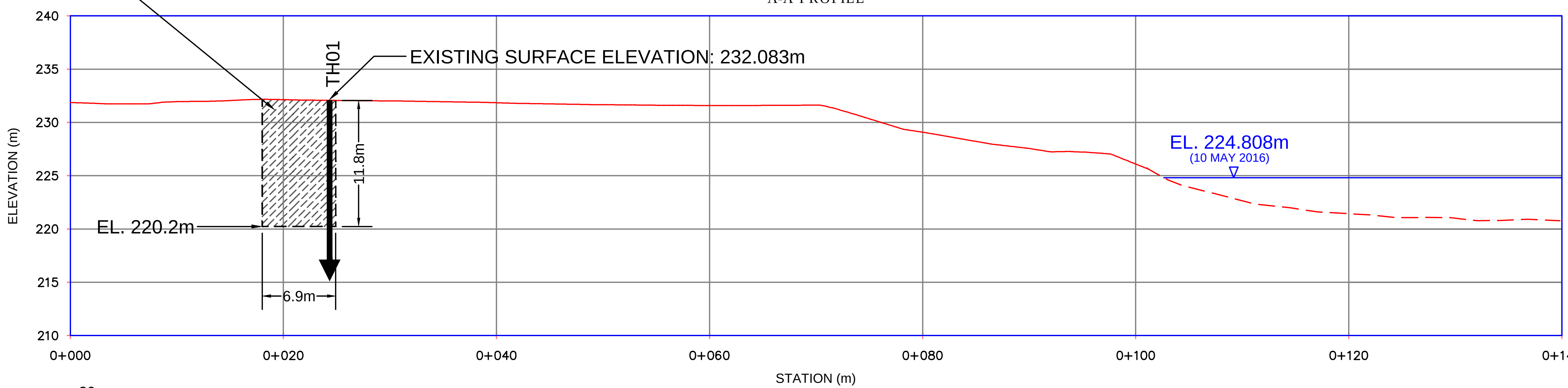
SITE AND TEST HOLE LOCATION PLAN

DATE:	JUNE 2016
PROJECT NO:	WX17932
REV. NO.:	A
FIGURE NO:	FIGURE 1

P:\JOBS\17900'S\17930'S\WX17932 – RUBY & AUBREY OUTFALL CHAMBERS\DRAWINGS\WX17932.DWG

PROPOSED RUBY
OUTFALL CHAMBER

A-A PROFILE



0 15 30 Meters

- IMAGE FROM http://winnipeg.ca/ppd/maps_aerial.stm

LEGEND:

RUBY SITE TEST HOLE — TH01

MMM PROFILE (05 MAY 2016 SURVEY)

ESTIMATED CHANNEL PROFILE FROM HEC-RAS (2011 BATHYMETRIC & LIDAR SURVEY DATA)

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

CLIENT:

MMM GROUP LIMITED

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

amec foster wheeler

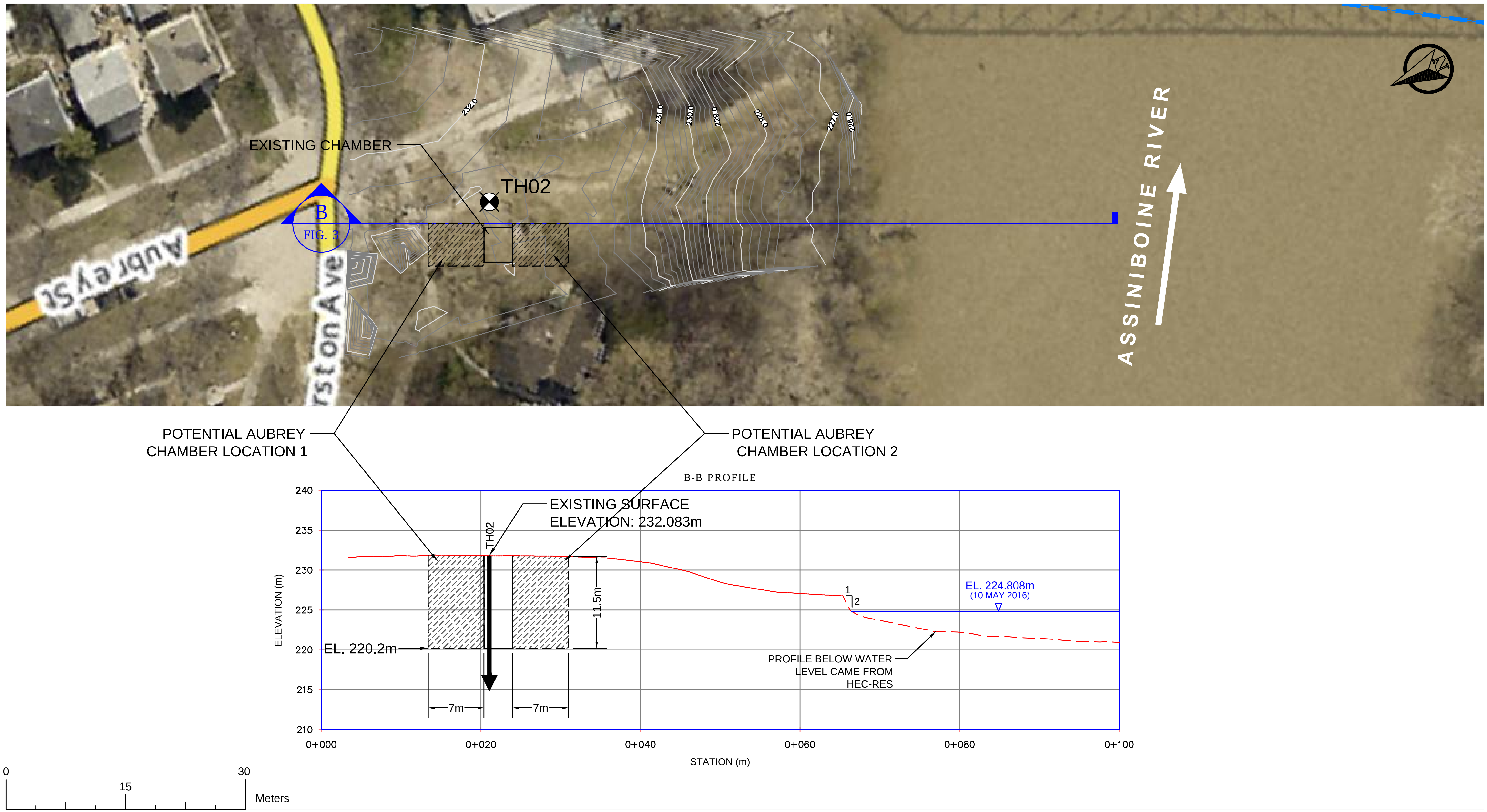
DWN BY: MD
CHK'D BY: WKW
DATUM: NAD83
PROJECTION: UTM Zone 14 U
SCALE: AS SHOWN

GEOTECHNICAL INVESTIGATION
RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE
WINNIPEG, MANITOBA

RIVERBANK PROFILE FOR THE PROPOSED RUBY SITE CHAMBER OUTFALL

DATE: JUNE 2016
PROJECT NO: WX17932
REV. NO.: A
FIGURE NO: FIGURE 2

P:\JOBS\17900'S\17930'S\WX17932 – RUBY & AUBREY OUTFALL CHAMBERS\DRAWINGS\WX17932.DWG



- IMAGE FROM http://winnipeg.ca/ppd/maps_aerial.stm

LEGEND:

AUBREY SITE TEST HOLE — TH02

MMM PROFILE (05 MAY 2016 SURVEY)

ESTIMATED CHANNEL PROFILE FROM HEC-RAS (2011 BATHYMETRIC & LIDAR SURVEY DATA)

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

CLIENT: MMM GROUP LIMITED

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: WKW

DATUM: NAD83

PROJECTION: UTM Zone 14 U

SCALE: AS SHOWN

GEOTECHNICAL INVESTIGATION
RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE
WINNIPEG, MANITOBA

RIVERBANK PROFILE FOR THE PROPOSED AUBREY SITE OUTFALL CHAMBER

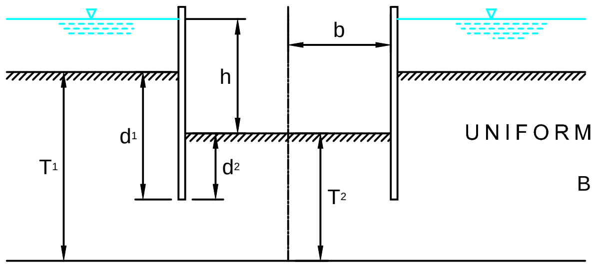
DATE: JUNE 2016

PROJECT NO: WX17932

REV. NO.: A

FIGURE NO: FIGURE 3

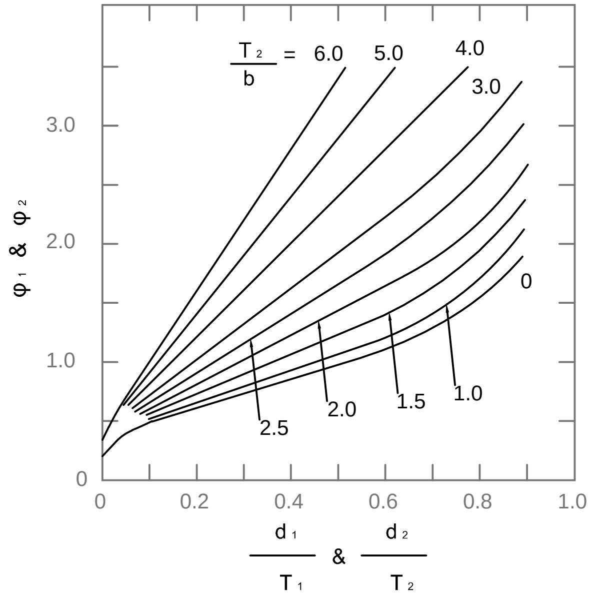
P: \JOBS\17900'S\17930'S\WX17932 – RUBY & AUBREY OUTFALL CHAMBERS\DRAWINGS\WX17932.DWG



(a)

UNIFORM SIZE SOIL

B = 2b



(b)

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 SQUARE EXCAVATION

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

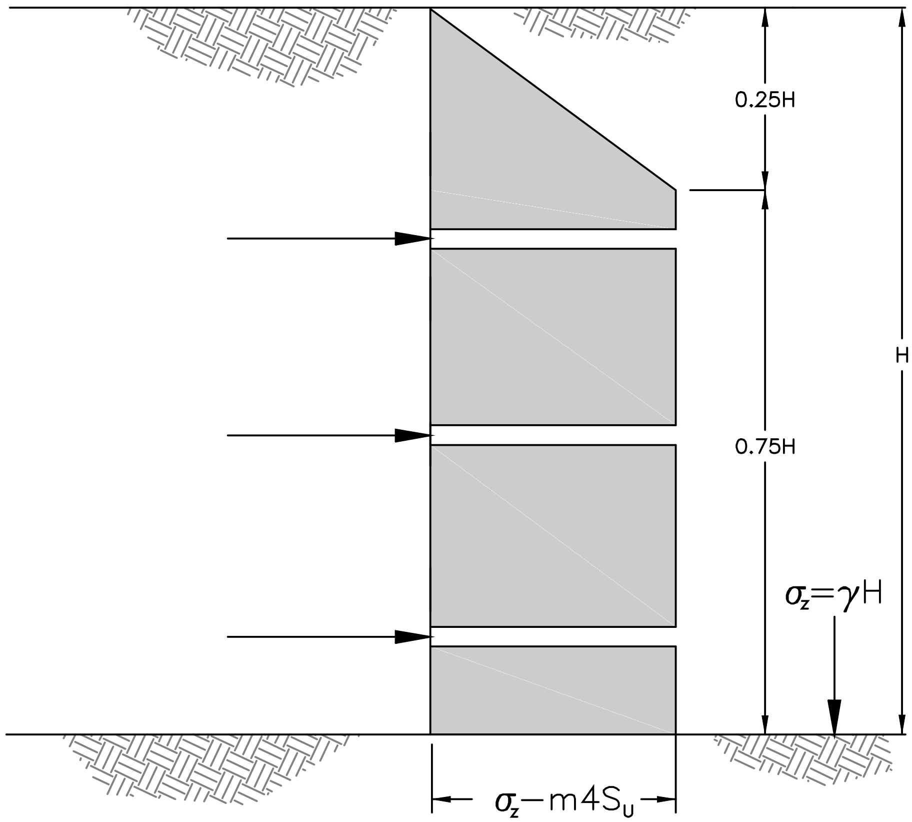
$$i_{exit} = 1.7 \frac{h}{d_2} \times \frac{\varphi_2}{\varphi_1 + \varphi_2}$$ (IN THE CORNERS)

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

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

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

MMM GROUP LIMITED	DWN BY: MD CHK'D BY: WKW	GEOTECHNICAL INVESTIGATION RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE WINNIPEG, MANITOBA	DATE: **JUNE 2016** PROJECT No.: WX17932
Amec Foster Wheeler Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	DATUM: - PROJECTION: - SCALE: NOT TO SCALE	ASSESSMENT OF SEEPAGE EXIT GRADIENT	REV. No.: ---- FIGURE No.: FIGURE 4



FOR CUTS WITH $\gamma H/S_u > 4$

WHERE: P_h = DESIGN LATERAL PRESSURE MAGNITUDE (kPa)

γ = BULK UNIT WEIGHT OF SOIL (USE γ =19kN/m)

H = HEIGHT OF WALL FROM FINISHED GRADE TO BASE OF WALL (m)

m = 0.4

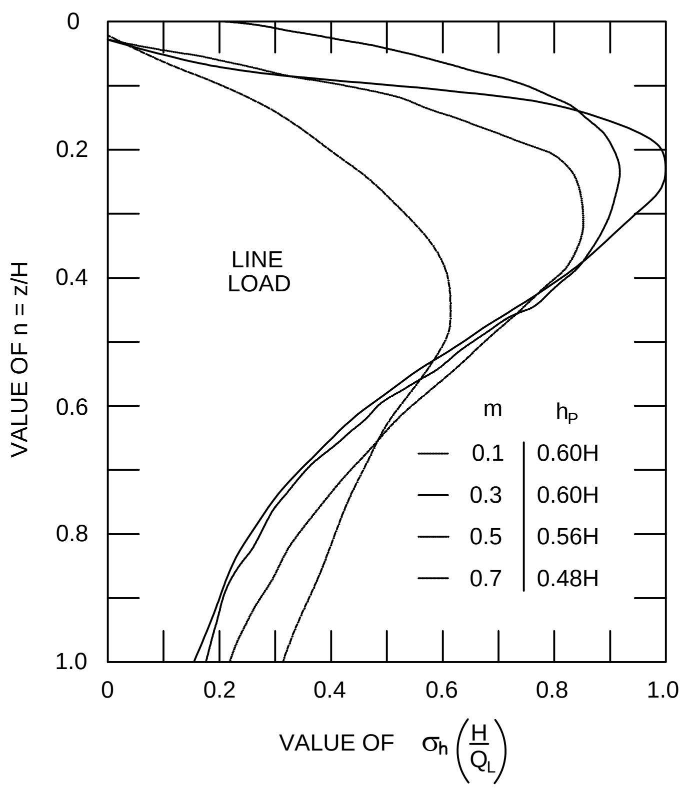
S_U = AVERAGE UNDRAINED SHEAR STRENGTH (USE 50kPa)

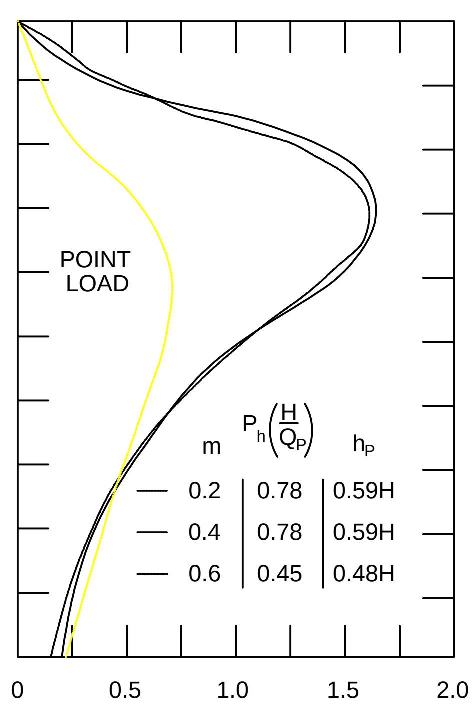
NOTES:

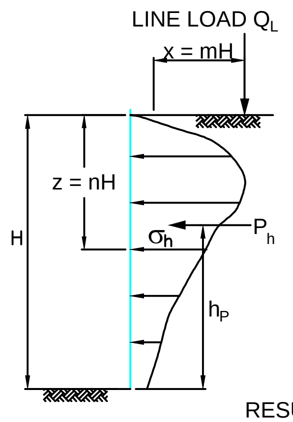
1. LATERAL PRESSURE DUE TO SURCHARGE LOAD AT THE SURFACE MUST BE ADDED TO THE ABOVE PRESSURE DIAGRAM.

P:\JOBS\17900'S\17930'S\WX17932 - RUBY & AUBREY OUTFALL CHAMBERS\DRAWINGS\GEO STANDARD FIGURES 4 AND 5.DWG

MMM GROUP LIMITED	DWN BY: MD CHK'D BY: WKW	GEOTECHNICAL INVESTIGATION RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE WINNIPEG, MANITOBA	DATE: JUNE 2016 PROJECT No.: WX17932
Amec Foster Wheeler Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	DATUM: - PROJECTION: - SCALE: NOT TO SCALE	LATERAL EARTH PRESSURE DISTRIBUTION FOR BRACED SHORING ON SOFT TO FIRM COHESIVE SOILS	REV. No.: ---- FIGURE No.: FIGURE 5







FOR m ≤ 0.4:

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

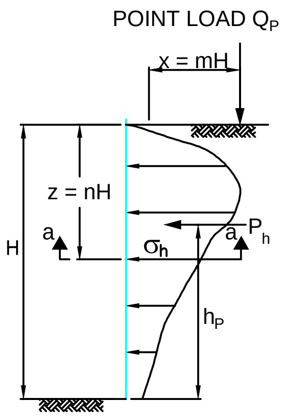
$P_h = 0.55\ Q_L$

FOR m > 0.4:

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

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

PRESSURES FROM LINE LOAD
(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



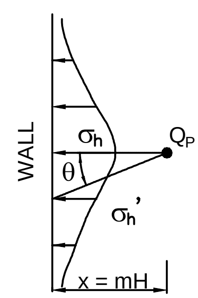
FOR m ≤ 0.4:

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

FOR m > 0.4:

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

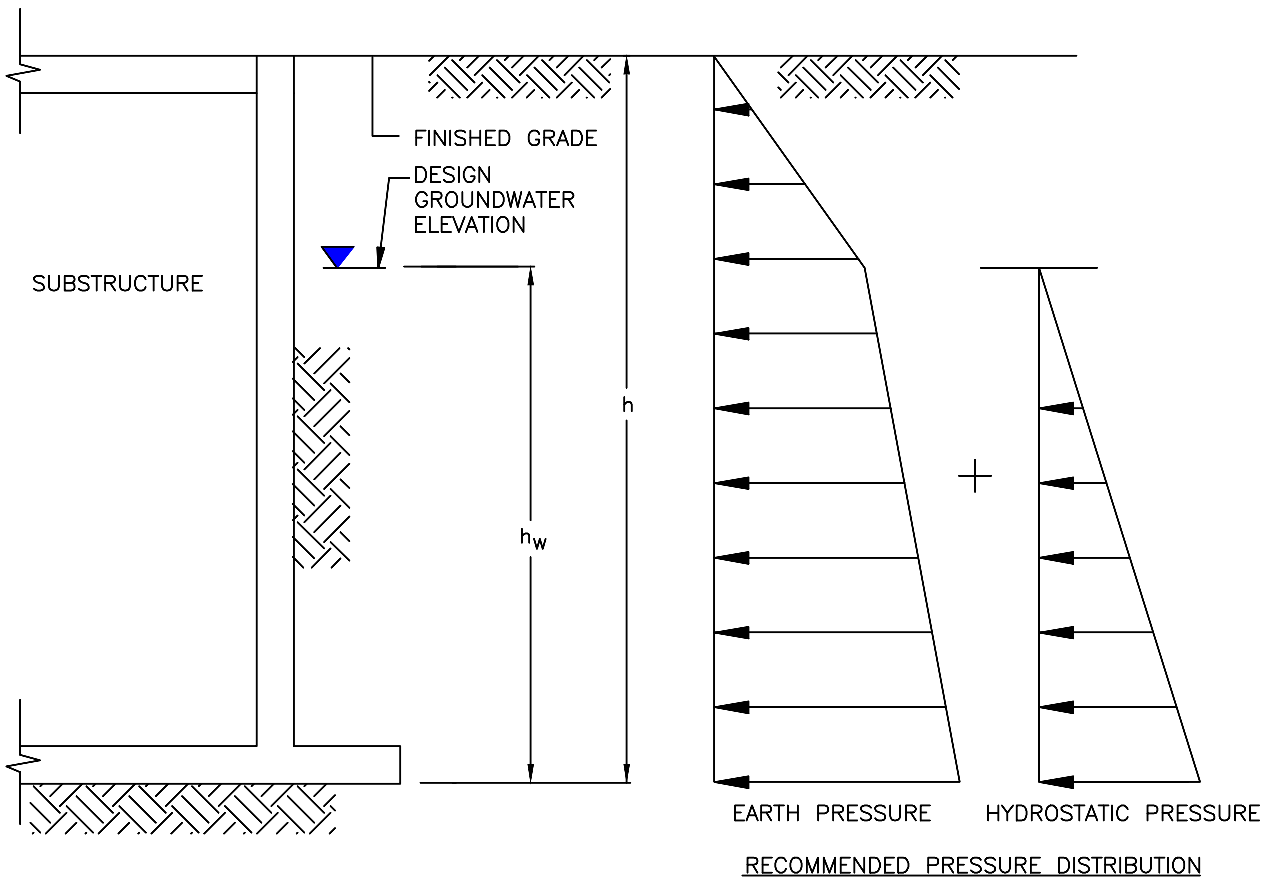
$$\sigma_h' = \sigma_h \cos^2(1.1\theta)$$



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

MMM GROUP LIMITED	DWN BY: MD CHK'D BY: WKW DATUM: -	GEOTECHNICAL INVESTIGATION RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE WINNIPEG, MANITOBA	DATE: JUNE 2016 PROJECT No.: WX17932 REV. No.: ----
Amec Foster Wheeler Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	PROJECTION: - SCALE: NOT TO SCALE	LATERAL PRESSURES DUE TO SURCHARGE POINT AND LINE LOADS	FIGURE No.: FIGURE 6

P:\JOBS\17900'S\17930'S\WX17932 - RUBY & AUBREY OUTFALL CHAMBERS\DRAWINGS\GEO STANDARD FIGURES 4 AND 5.DWG



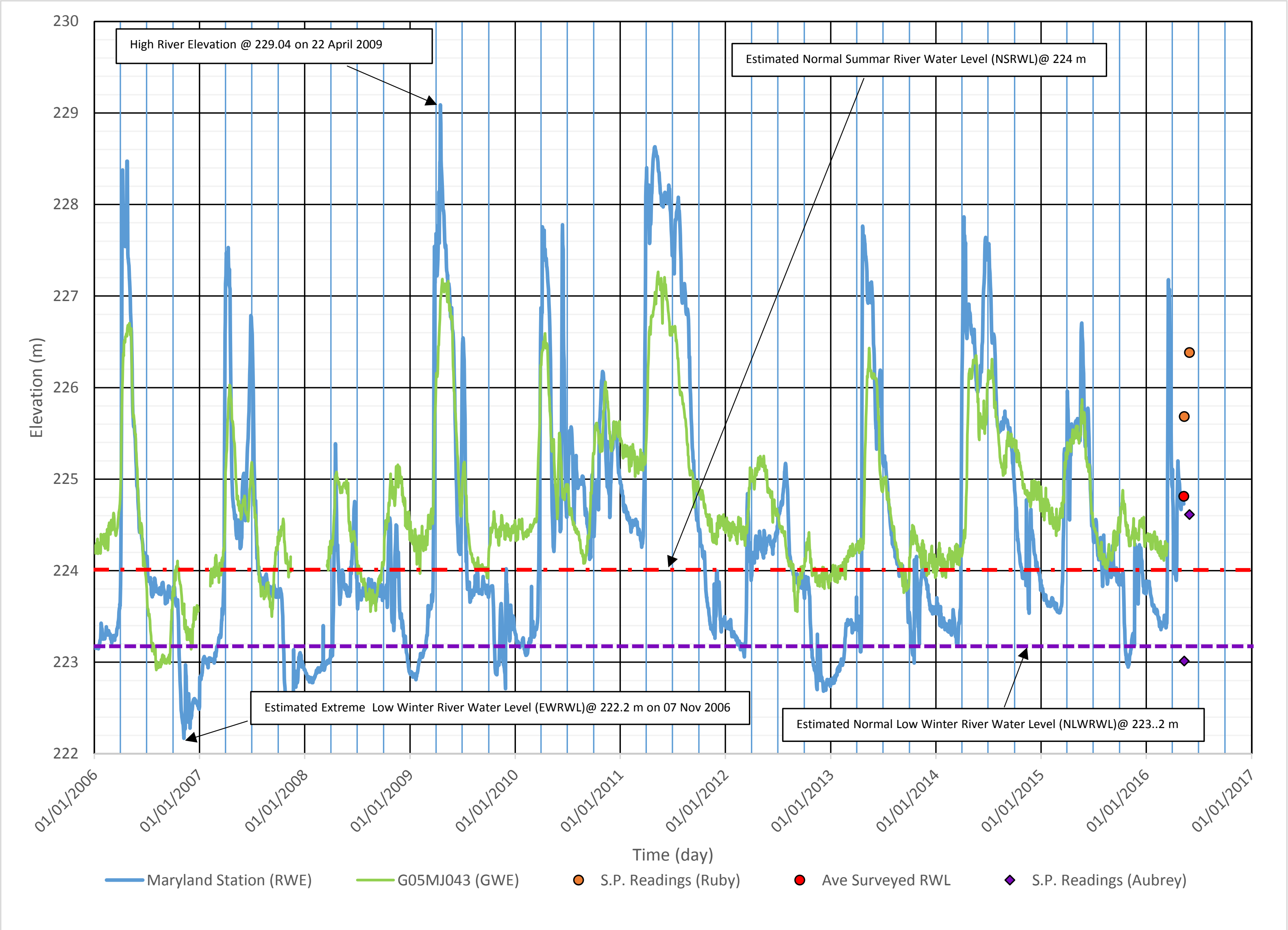
$$P_h = K\gamma(h-h_w) + K\gamma' h_w + \gamma_w h_w$$

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_a OR 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.

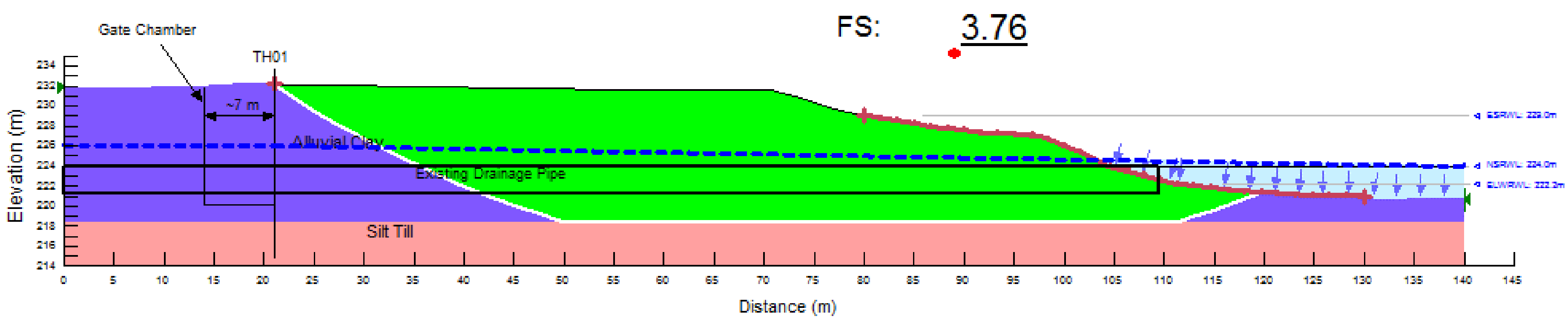
MMM GROUP LIMITED	DWN BY: MD	GEOTECHNICAL INVESTIGATION RUBY AND AUBREY OUTFALL CHAMBERS UPGRADE WINNIPEG, MANITOBA	DATE: **JUNE 2016**
	CHK'D BY: WKW		PROJECT No.: WX17932
Amec Foster Wheeler Environment & Infrastructure 440 DOVERCOURT DRIVE WINNIPEG, MANITOBA R3Y 1N4 PHONE: 204.488.2997 FAX:204.489.8261	DATUM: -	LATERAL EARTH PRESSURES ON PERMANENT GATE CHAMBER WALLS	REV. No.: ----
	PROJECTION: -		FIGURE No.: FIGURE 7
	SCALE: NOT TO SCALE		



amec foster wheeler	Client: MMM Group	Historical Assiniboine River Elevations and Groundwater Elevations Proposed Ruby and Aubrey Outfall Gate Chamber Upgrades 980 & 1016 Palmerton Avenue, Winnipeg, Manitoba	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	8

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

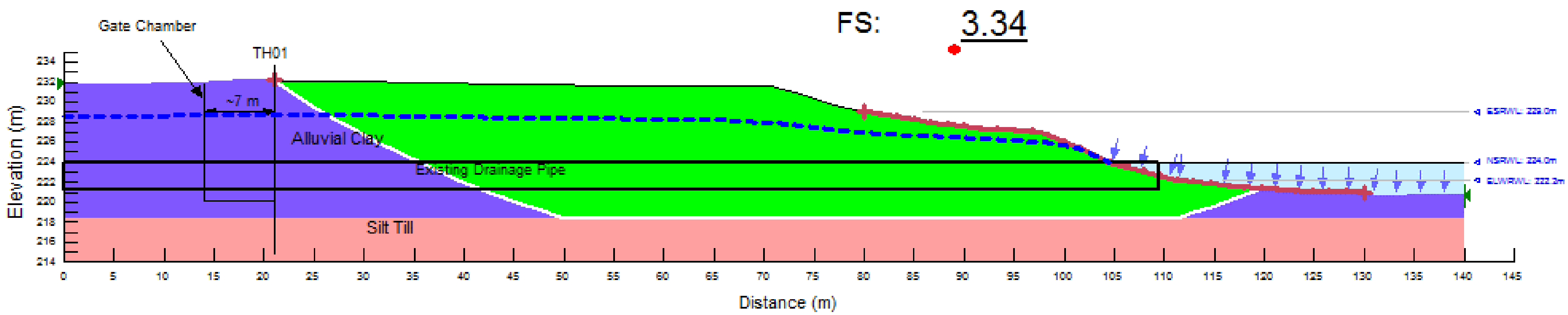


File Name: WX17932 - RUBY.gsz
Name: Summer (Normal) - NSWL
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Normal Summer Design Conditions** **Proposed Ruby Outfall Gate Chamber Upgrades** **980 Palmerton Avenue,** **Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	9

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

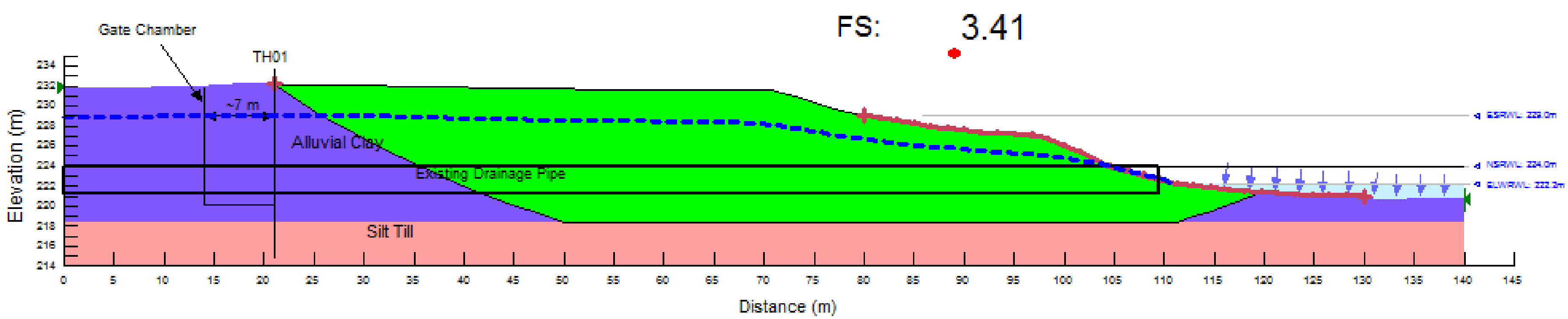


File Name: WX17932 - RUBY.gsz
Name: Summer (Extreme) - ESWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Spring Drawdown Extreme Design Conditions** **Proposed Ruby Outfall Gate Chamber Upgrades** **980 Palmerton Avenue,** **Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	10

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till



File Name: WX17932 - RUBY.gsz
Name: Winter (Extreme) - EWWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler

Client:

MMM Group

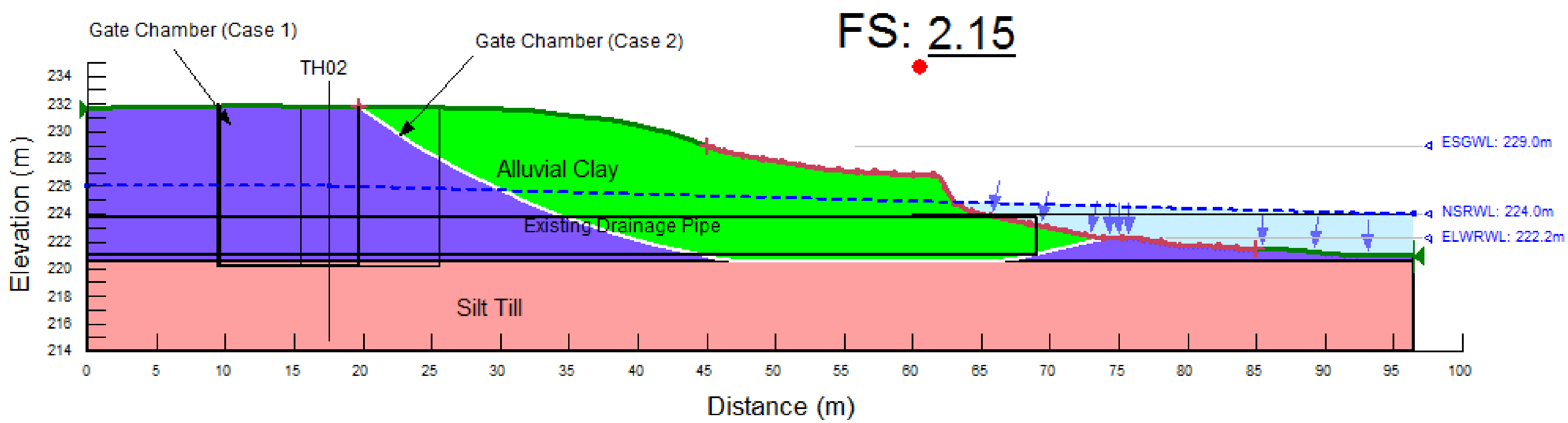
Fall Drawdown Extreme Design Conditions
Proposed Ruby Outfall Gate Chamber Upgrades

980 Palmerton Avenue,
Winnipeg, Manitoba

Project No.:	WX17932
Date:	June 2016
Scale:	As Shown
Drawn By:	WKW
Figure:	11

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

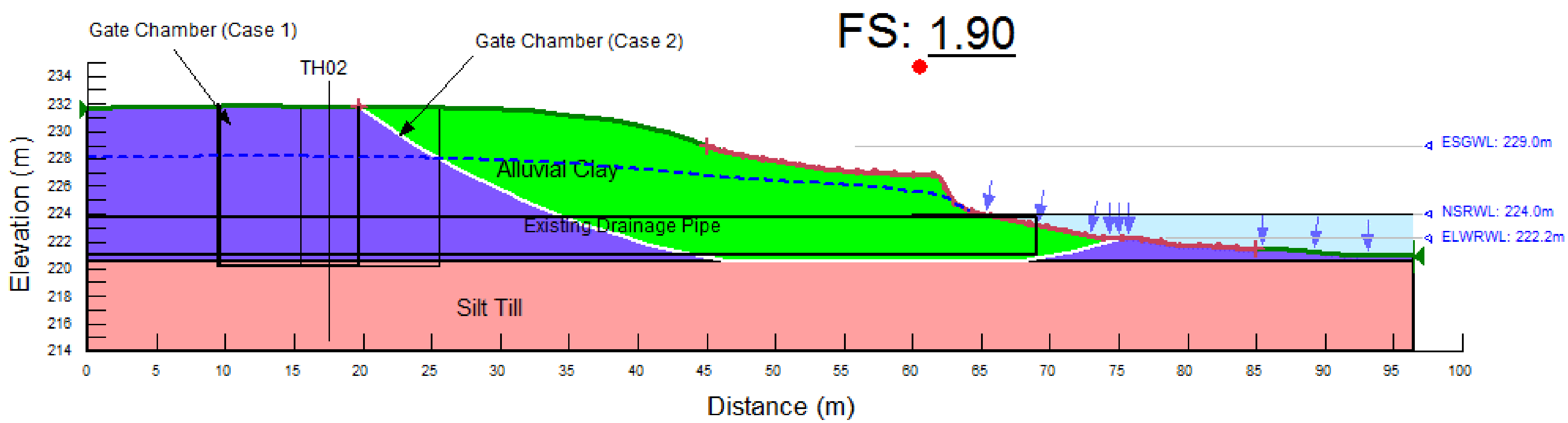


File Name: WX17932 - AUBREY.gsz
Name: Case1 - Summer (Normal) - NSWL
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 1: Normal Summer Design Conditions** **Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue,** **Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	12

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

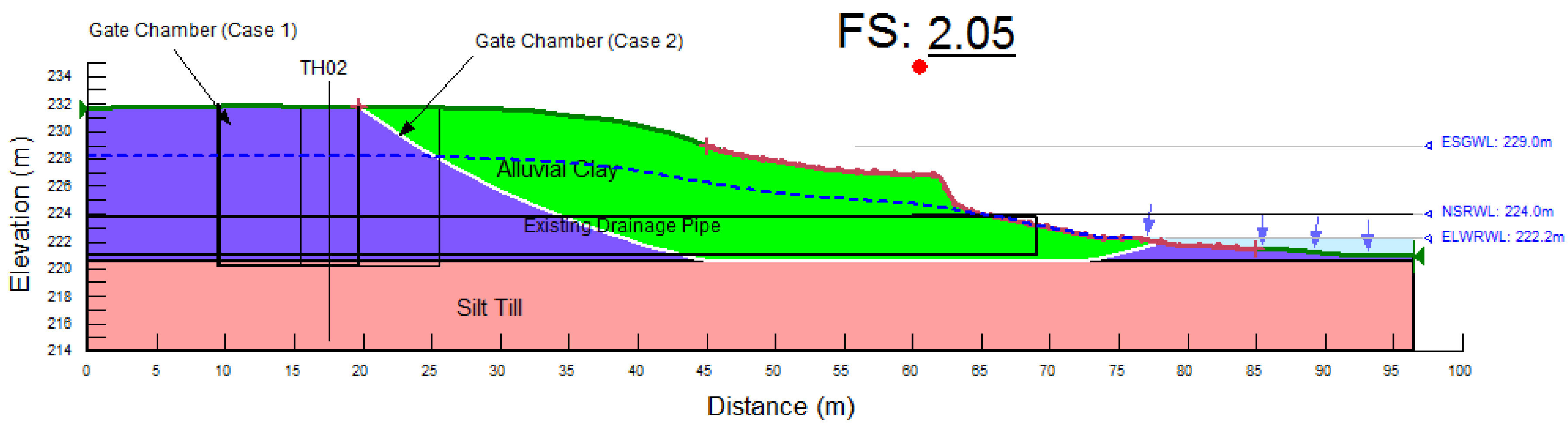


File Name: WX17932 - AUBREY.gsz
Name: Case1 - Summer (Extreme) - ESWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 1: Spring Drawdown Extreme Design Conditions** **Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue, Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	13

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

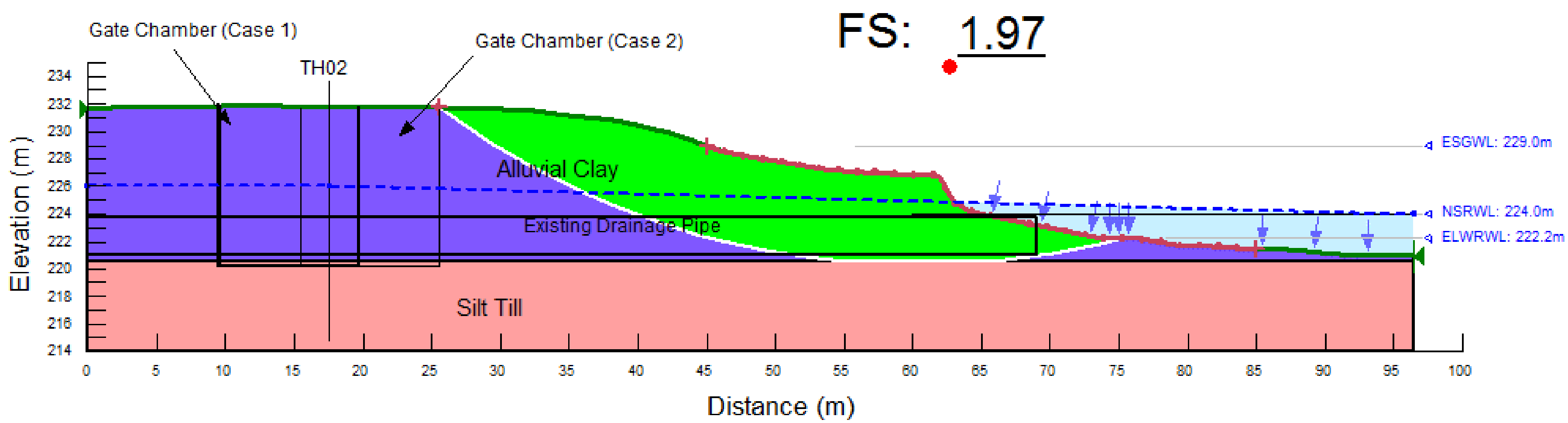


File Name: WX17932 - AUBREY.gsz
Name: Case1 - Winter (Extreme) - EWWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 1: Fall Drawdown Extreme Design Conditions** **Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue,** **Winnipeg, Manitoba**	Project No.: WX17932 Date: June 2016 Scale: As Shown Drawn By: WKW Figure: 14

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

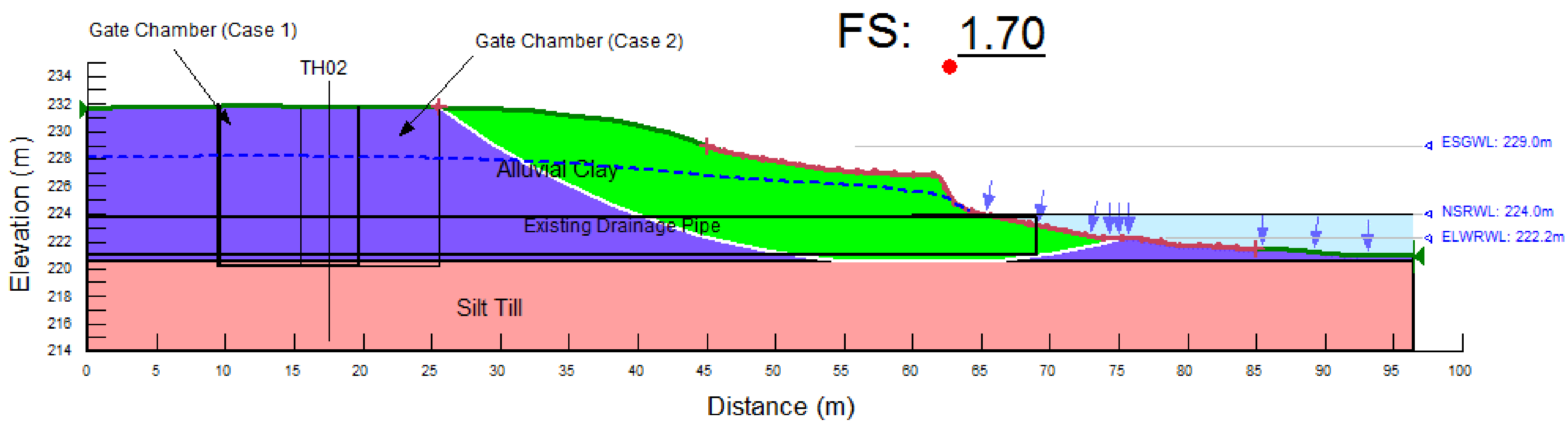


File Name: WX17932 - AUBREY.gsz
Name: Case2 - Summer (Normal) - NSWL
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 2: Normal Summer Design Conditions Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue, Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	15

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till

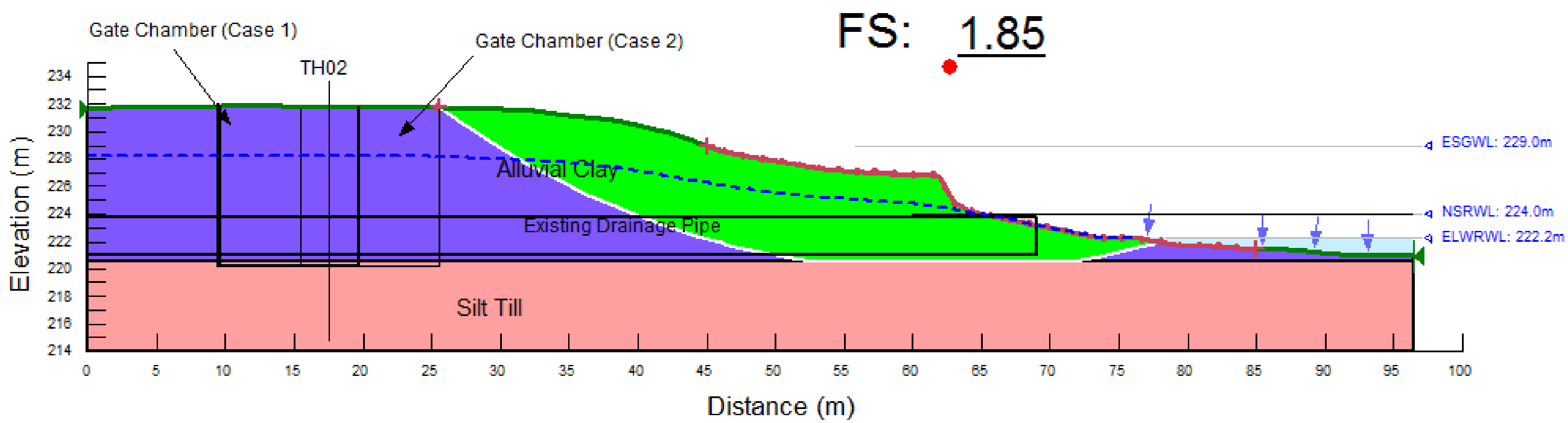


File Name: WX17932 - AUBREY.gsz
Name: Case2 - Summer (Extreme) - ESWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 2: Spring Drawdown Extreme Design Conditions Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue, Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	16

Name: Alluvial Clay
Unit Weight: 19 kN/m³
Cohesion': 3 kPa
Phi': 23 °

Name: Silt Till



File Name: WX17932 - AUBREY.gsz
Name: Case2 - Winter (Extreme) - EWWL (Seep)
Directory: P:\Jobs\17900's\17930's\WX17932 - Ruby & Aubrey Outfall Chambers\Riverbank Stability\

amec foster wheeler	**Client:** **MMM Group**	**Case 2: Fall Drawdown Extreme Design Conditions** **Proposed Aubrey Outfall Gate Chamber Upgrades** **1016 Palmerton Avenue,** **Winnipeg, Manitoba**	Project No.:	WX17932
			Date:	June 2016
			Scale:	As Shown
			Drawn By:	WKW
			Figure:	17

APPENDIX A

Soil Logs

EXPLANATION OF TERMS AND SYMBOLS

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

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

TEST DATA

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

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

*C	Consolidation test	*ST	Swelling test
D_R	Relative density	TV	Torvane shear strength
*k	Permeability coefficient	VS	Vane shear strength
*MA	Mechanical grain size analysis and hydrometer test	w	Natural Moisture Content (ASTM D2216)
		w_l	Liquid limit (ASTM D 423)
N	Standard Penetration Test (CSA A119.1-60)	w_p	Plastic Limit (ASTM D 424)
N_d	Dynamic cone penetration test	E_f	Unit strain at failure
NP	Non plastic soil	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	$C_u=D_{60}/D_{10} > 4$; $C_c=(D_{30})^2/(D_{10} x D_{60})$ = 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	$C_u=D_{60}/D_{10} > 6$; $C_c=(D_{30})^2/(D_{10} x D_{60})$ = 1 to 3
			SP		RED	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS
		DIRTY SANDS (WITH SOME OR MORE FINES)	SM		YELLOW	SILTY SANDS, SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4
			SC		YELLOW	CLAYEY SANDS, SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE AND PI MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT SMALLER THAN 75um)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)
		$W_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SAND OR SILTY SOILS	
	CLAYS ABOVE "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY OR SILTY CLAYS, LEAN CLAYS	
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS	
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	ORGANIC SILTS & CLAYS BELOW "A" LINE	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINES CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGHLY ORGANIC SOILS			PT		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE

SPECIAL SYMBOLS

LIMESTONE		OILSAND	
SANDSTONE		SHALE	
SILTSTONE		FILL (UNDIFFERENTIATED)	

SOIL COMPONENTS

FRACTION	U.S. STANDARD METRIC SIEVE SIZE: PASSING	U.S. STANDARD METRIC SIEVE SIZE: RETAINED	DEFINING RANGES OF PERCENT BY WEIGHT OF MINOR COMPONENTS: PERCENT	DESCRIPTOR
GRAVEL				
COARSE	76mm	19mm		
FINE	19mm	4.75mm	35 - 50	AND
SAND				
COARSE	4.75mm	2.00mm	30 - 35	Y / EY
MEDIUM	2.00mm	425μm		
FINE	425μm	75μm	10 - 20	SOME
FINES (SILT OR CLAY BASED ON PLASTICITY)	75μm		1 - 10	TRACE

OVERSIZED MATERIAL

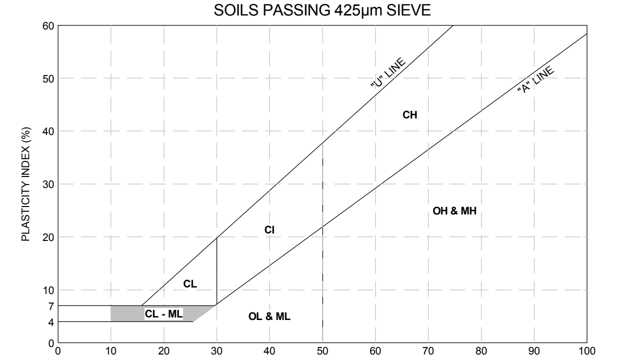
ROUNDED OR SUBROUNDED:

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



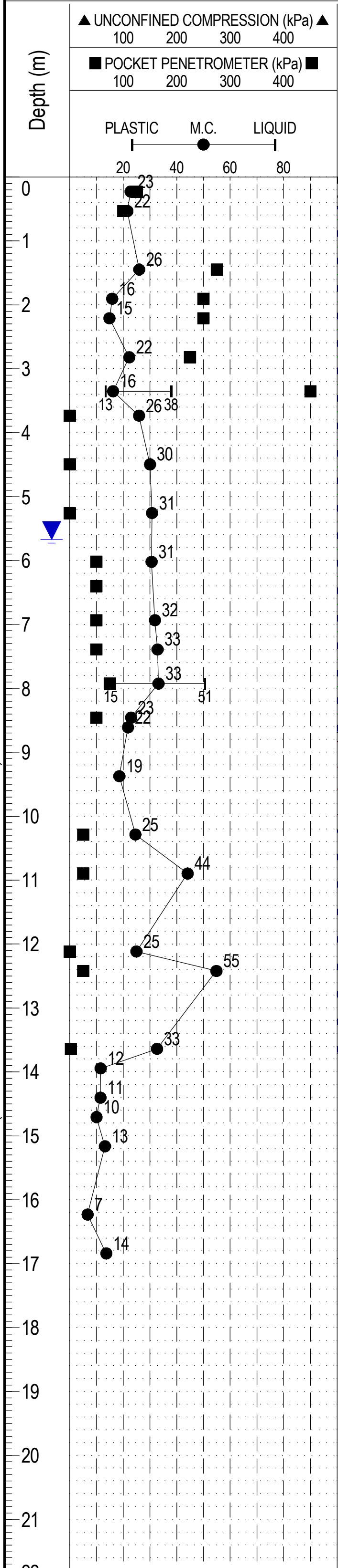
NOTES:
1. ALL SIEVE SIZES MENTIONED ARE U.S. STANDARD ASTM E.11.
2. COARSE GRAINED SOILS WITH TRACE TO SOME FINES GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH TRACE TO SOME CLAY.
3. DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

amec foster wheeler

PROJECT: Ruby & Aubrey Outfall Chambers	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH01 (Ruby)**
CLIENT: MMM Group Limited	DRILL TYPE: Renegade Track Rig	PROJECT NO: WX17932
LOCATION: 5526579 mN, 631231 mE	DRILL METHOD: 125mm SSA	ELEVATION: 232.08 m

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

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



Depth (m)	MUSCS	SOIL DESCRIPTION
0	ASPH	ASPHALT - Approximate 90mm thick
	CI-CH	CLAY (FILL) - silty, some sand, trace gravel, medium to high plastic, moist, stiff, dark greyish brown
	CI-CH	CLAY (ALLUVIAL) - silty, trace to some sand, medium to high plastic, moist, stiff, dark greyish brown mottled dark grey, occasional sand lenses - and sand, greyish brown below 1.8m - brown, occasional oxidized inclusions below 2.7m - trace to some sand, silty to and silt, very soft, very moist to wet, occasional to frequent silt pockets (~5mm thick) from 3.7 to 6.1m - trace gravel, firm to stiff, dark greyish brown, frequent oxidized inclusions below 6.1m - silty, some sand, moist, dark grey, occasional fine sand seams (<3mm thick), frequent oxide pockets (<2cm thick) below 7.3m - sandy, grey below 8.2m
	SM	SAND - silty, poorly graded, fine grained, wet, compact, brown
	CH	CLAY (ALLUVIAL) - silty, high plastic, very moist, soft to firm, dark grey, occasional sand inclusions, occasional silt and sulphate inclusions - trace to some gravel, occasional sand pockets (~5cm to 10cm dia.) below 11.1m - wet, very soft below 12.5m
	ML	SILT (TILL) - sandy, some gravel, trace clay, low plastic, wet, compact, light greyish brown - gravelly below 14.2m
		AUGER REFUSAL AT 16.9m BELOW GRADE. Notes: - Seepage & sloughing observed at zones from 4.3m to 6.1m, from 8.5m to 10.2m and below 13.7m during and on completion of drilling. - Test hole remained open to 14.6m and water level at 6.4m below grade was observed prior to backfilling.\ - One 25mm diameter standpipe installed on completion of drilling.

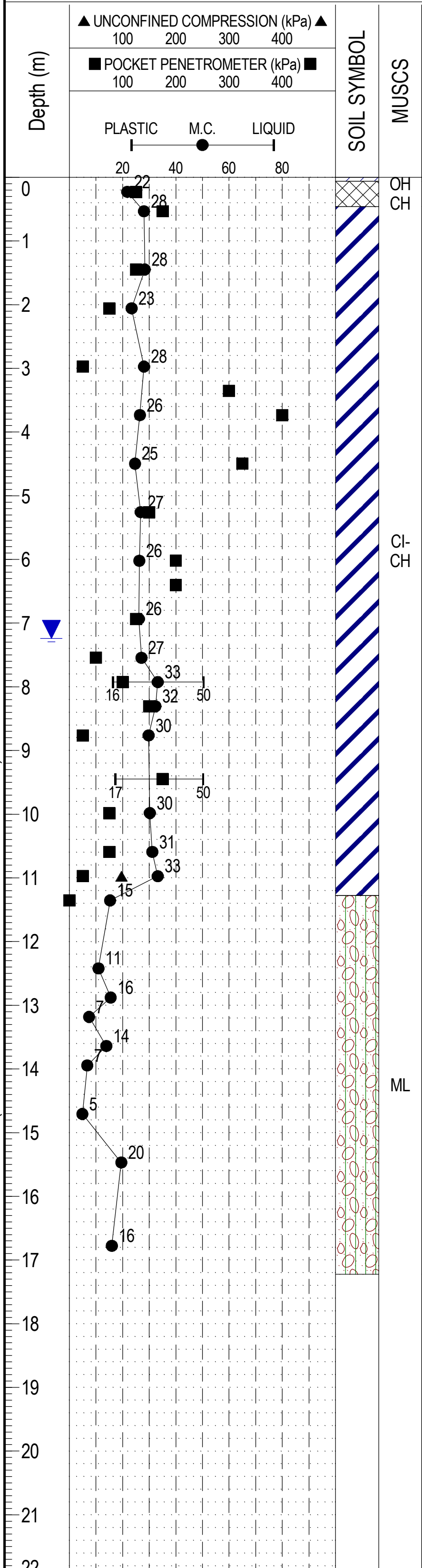
SAMPLE NO	SAMPLE TYPE	SPT (N)	COMMENTS
1	Grab		
2	Grab		
3	Grab		Ground water level: - 6.4m below ground surface on 12/05/2016 - 5.7m below ground surface on 31/05/2016
4	Grab		
5	Grab		
6	Grab		
7	Shelby Tube		Hydrometer Analysis Results @ "3.0m": Gravel= 0.0% Sand= 23.4% Silt= 46.8% Clay= 29.8%
8	Grab		
9	Grab		
10	Grab		Unconfined Compression Test: Sample 7 (3.0m - 3.6m) Max Stress: 270.6 kPa @ 8.7% strain M.C: 16.2%
11	Grab		
12	Shelby Tube		
13	SPT	3	..SPT: 1,1,2; Rec: full
14	Grab		
15	Shelby Tube		Unconfined Compression Test: Sample 15 (7.6m - 8.2m) Max Stress: 97.7 kPa @ 5.8% strain M.C: 33.2%
16	Grab		
17	Grab		
18	SPT	14	..SPT: 4,7,7; Rec: full
19	Grab		
20	SPT	4	..SPT: 2,2,2; Rec: 230mm
21	Grab		
22	SPT	5	..SPT: 3,2,3; Rec: 150mm
23	Grab		
24	SPT	27	..SPT: 2,15,12; Rec: 150mm
25	Grab		
26	SPT	12	..SPT: 8,4,8; Rec: 230mm
27	Grab		
28	SPT	24	..SPT: 8,18,6; Rec: 50mm
29	Grab		

ELEVATION (m): 231, 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211

Amec Foster Wheeler **Winnipeg, Manitoba**	LOGGED BY: KE	COMPLETION DEPTH: 16.9 m
	REVIEWED BY: WKW	COMPLETION DATE: 12 May 2016
	Figure No. A01	Page 1 of 1

PROJECT: Ruby & Aubrey Outfall Chambers	DRILLED BY: Maple Leaf Drilling Ltd.	BORE HOLE NO: **TH02 (Aubrey)**
CLIENT: MMM Group Limited	DRILL TYPE: Renegade Track Rig	PROJECT NO: WX17932
LOCATION: 5526584 mN, 631083 mE	DRILL METHOD: 125mm SSA	ELEVATION: 231.81 m

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



SOIL DESCRIPTION	SAMPLE NO	SPT (N)	COMMENTS
ORGANIC CLAY (TOPSOIL) - silty, trace sand, high plastic, moist	1, 2		
CLAY (FILL) - silty, trace sand, trace gravel, high plastic, moist, stiff, dark greyish brown mottled light brown, occasional oxidized inclusions	3		Ground water level: - 8.8m below ground surface on 12/05/2016 - 7.2m below ground surface on 31/05/2016
CLAY (ALLUVIAL) - silty, trace sand, high plastic, moist, stiff, dark greyish brown, occasional silt inclusions, trace organics, occasional black stains (5mm to 10mm thick) - brown, silty to and silt, firm to soft below 2.0m - silty, sandy and sand, medium plastic, moist, very stiff below 3.0m - high plastic, dark greyish brown, occasional silt lenses (1mm to 2mm thick), occasional oxidized inclusions below 3.7m	4, 5, 6, 7, 8, 9		
- medium to high plastic, moist, stiff, dark greyish brown, some oxidation below 5.0m - wet sand lenses (~1mm to 5mm thick) at 6.7m	10, 11, 12	10	..SPT: 4,4,6; Rec: 410mm
- some sand to sandy, very moist, firm, dark brownish grey, trace oxidation below 7.6m - stiff from 7.6 to 8.7m - grey to dark grey below 8.2m - very moist, soft at 8.7m - sandy and sand, medium plastic, firm below 9.1m	13, 14, 15, 16, 17, 18	5	Unconfined Compression Test: Sample 14 (7.6m - 8.2m) Max Stress: 101.0 kPa @ 9.8% strain M.C: 33.2% Hydrometer Analysis Results @ "9.1m": Gravel= 0.0% Sand= 5.8% Silt= 59.7% Clay= 34.5% ..SPT: 2,2,3; Rec: full
- some sand, high plastic, moist to very moist, firm, dark grey, frequent light grey till inclusions (1 to 5 cm dia) below 10.7m	19, 20, 21		Unconfined Compression Test: Sample 20 (10.6m - 11.2m) Max Stress: 57.2 kPa @ 6.3% strain M.C: 33.3%
SILT (TILL) - clayey, some sand, trace gravel, low plastic, compact, very moist to wet, light brown - some clay, some gravel to gravelly, wet below 12.2m	22, 23, 24, 25, 26	20, 18, 16	..SPT: 2,9,11; Rec: 300mm ..SPT: 9,9,9; Rec: 50mm ..SPT: 5,9,7; Rec: 100mm
- gravelly, wet, light greyish brown, occasional sand lenses below 14.5m	27, 28, 29	23, 8	..SPT: 10,12,11; Rec: 50mm ..SPT: 5,5,3; Rec: 50mm ..SPT: 10/50mm

AUGER REFUSAL AT 17.2m BELOW GRADE.
Notes:
- Seepage & sloughing observed below 11.3 m on completion of drilling.
- Seepage observed at the sand lenses at 6.7 m during drilling.
- Test hole remained open to 17.2m and water level at 8.8m below grade was observed prior to backfilling.
- One 25mm diameter standpipe installed on completion of drilling.

ELEVATION (m): 231 230 229 228 227 226 225 224 223 222 221 220 219 218 217 216 215 214 213 212 211 210

Amec Foster Wheeler Winnipeg, Manitoba	LOGGED BY: KE	COMPLETION DEPTH: 17.2 m
	REVIEWED BY: WKW	COMPLETION DATE: 12 May 2016
	Figure No. A02	Page 1 of 1

APPENDIX B

Laboratory Testing Results

UNCONFINED COMPRESSION

IN ACCORDANCE WITH ASTM D2166

Figure B1

TO: MMM Group

OFFICE: Winnipeg
PROJECT NO: WX17932
COPIES TO:
TEST DATE: May 24, 2016

ATTENTION:

PROJECT: Ruby & Aubrey Outfall Gate Chambers

TEST HOLE: 1 **SAMPLE #:** 7
TECHNICIAN: VM **DEPTH:** 10-12ft

	Yes	No
ATTERBERG LIMITS:		X
HYDROMETER:		X

WET WEIGHT: 989.6 g
MOISTURE (%): 16.2

SAMPLE DIAMETER: 71.94 mm, 71.98 mm, 72.02
AVERAGE: 71.98 mm

INITIAL AREA: 4069.24 mm^2
LENGTH: 116.77 mm, 116.75 mm
AVERAGE: 116.76 mm
VOLUME: 0.000475 m^3

DENSITY
WET: **2082.8** (kg/m^3)
DRY: **1792.3** (kg/m^3)

STRAIN RATE: 1.11 **%/min.**
(to failure) (0.5% to 2% per min as per ASTM)

Load Dial (0.0001")	Elapsed Time (min)	Strain Dial (0.001")	Strain %	Axial Load (2014 Data)	Stress Load (kPa)
68	0.5	31	0.53	98.9	**24.2**
160	1.0	62	1.00	211.2	**51.4**
218	1.5	90	1.48	287.7	**69.7**
280	2.0	120	2.00	371.7	**89.5**
342	2.5	149	2.50	455.7	**109.2**
410	3.0	177	2.96	542.8	**129.4**
480	3.5	206	3.44	630.9	**149.7**
550	4.0	235	3.92	723.8	**170.9**
583	4.5	263	4.45	818.2	**192.1**
614	5.0	291	4.99	911.2	**212.7**
641	5.5	320	5.57	996.8	**231.3**
662	6.0	350	6.17	1065.4	**245.7**
678	6.5	380	6.79	1118.9	**256.3**
690	7.0	410	7.42	1159.1	**263.7**
700	7.5	440	8.05	1192.5	**269.5**
704	8.0	470	8.69	1206.1	**270.6**
705	8.5	500	9.34	1209.5	**269.4**
700	9.0	530	10.01	1192.5	**263.7**

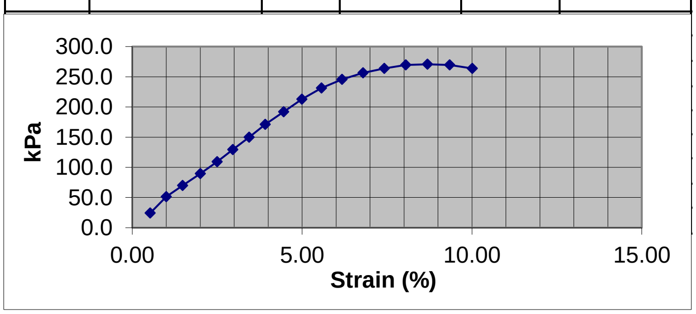


Photo of Failed Speciman

Max. Compressive Stress (kPa)
270.6

Pocket Pen kPa	Torvane kPa
Top	Top
375	70
400	75
410	80
375	70
390 Avg	**74** Avg
Bottom	Bottom
350	70
400	80
425	85
375	82
388 Avg	**79** Avg
	1 Torvane Factor

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.

Amec Foster Wheeler Environment & infrastructure

Per.

Trevor Gluck, P. Eng.
Manager; Technical Services

AMEC Environment & Infrastructure
A division of AMEC Americas Limited
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Winnipeg, Manitoba
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Rev 2014a

UNCONFINED COMPRESSION

IN ACCORDANCE WITH ASTM D2166

Figure B2

amec foster wheeler

TO: **MMM Group**

OFFICE: Winnipeg
PROJECT NO: **WX17932**
COPIES TO:
TEST DATE: **May 24, 2016**

ATTENTION:

PROJECT: **Ruby & Aubrey Outfall Gate Chambers**

TEST HOLE: **1** **SAMPLE #:** 15
TECHNICIAN: **MV/VS** **DEPTH:** 25-27ft

	Yes	No
ATTERBERG LIMITS:		**X**
HYDROMETER:		**X**

WET WEIGHT: 938.1 g
MOISTURE (%): 33.2

SAMPLE DIAMETER: 71.23 mm, 71.76 mm, 71.19
AVERAGE: 71.39 mm

INITIAL AREA: 4003.18 mm^2
LENGTH: 125.75 mm, 124.85 mm
AVERAGE: 125.30 mm
VOLUME: 0.000502 m^3

DENSITY
WET: **1870.2** (kg/m^3)
DRY: **1404.2** (kg/m^3)

STRAIN RATE: 1.06 **%/min.**
(to failure) (0.5% to 2% per min as per ASTM)

Load Dial (0.0001")	Elapsed Time (min)	Strain Dial (0.001")	Strain %	Axial Load (2014 Data)	Stress Load (kPa)
38	0.5	30	0.53	53.8	**13.4**
78	1.0	59	1.04	111.8	**27.6**
119	1.5	85	1.48	162.3	**39.9**
154	2.0	115	2.02	203.3	**49.8**
189	2.5	142	2.50	249.0	**60.7**
219	3.0	171	3.02	289.1	**70.0**
240	3.5	200	3.57	317.5	**76.5**
272	4.0	228	4.07	360.9	**86.5**
292	4.5	258	4.64	388.0	**92.4**
306	5.0	283	5.12	407.0	**96.5**
312	5.5	316	5.77	415.1	**97.7**
311	6.0	343	6.32	413.8	**96.8**
309	6.5	371	6.89	411.1	**95.6**

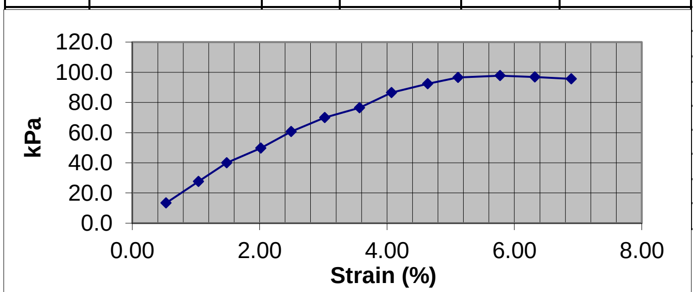

Photo of Failed Speciman

Max. Compressive Stress (kPa)
97.7

Pocket Pen kPa	Torvane kPa
Top	Top
75	48
75	38
75	38
100	38
81	**40**
Avg	Avg
Bottom	Bottom
75	30
100	30
75	28
75	28
81	**29**
Avg	Avg
	1
	Torvane Factor

Reporting of these test results constitutes a testing service only.
Engineering interpretation or evaluation of the test results is provided only on written request.

Amec Foster Wheeler Environment & infrastructure

Per.

Trevor Gluck, P. Eng.
Manager; Technical Services

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Rev 2014a

UNCONFINED COMPRESSION

Figure B3

IN ACCORDANCE WITH ASTM D2166

TO: **MMM Group**

OFFICE: Winnipeg
PROJECT NO: **WX17932**
COPIES TO:
TEST DATE: **May 24, 2016**

ATTENTION:

PROJECT: **Ruby & Aubrey Outfall Gate Chambers**

TEST HOLE: **2** **SAMPLE #:** 14
TECHNICIAN: **VS** **DEPTH:** 25-27ft

	Yes	No
ATTERBERG LIMITS:		**X**
HYDROMETER:		**X**

WET WEIGHT: 1158.8 g
MOISTURE (%): 33.2

SAMPLE DIAMETER: 71.73 mm, 71.50 mm, 71.37
AVERAGE: 71.53 mm

INITIAL AREA: 4018.90 mm^2
LENGTH: 154.09 mm, 154.29 mm
AVERAGE: 154.19 mm
VOLUME: 0.000620 m^3

DENSITY
WET: **1870.0** (kg/m^3)
DRY: **1403.6** (kg/m^3)

STRAIN RATE: 0.90 **%/min.**
(to failure) (0.5% to 2% per min as per ASTM)

Load Dial (0.0001")	Elapsed Time (min)	Strain Dial (0.001")	Strain %	Axial Load (2014 Data)	Stress Load (kPa)
37	0.5	28	0.40	52.5	**13.0**
67	1.0	55	0.80	97.6	**24.1**
100	1.5	84	1.22	140.3	**34.5**
135	2.0	115	1.67	180.8	**44.2**
165	2.5	142	2.07	217.7	**53.0**
188	3.0	171	2.51	247.7	**60.1**
208	3.5	200	2.95	274.2	**66.2**
224	4.0	229	3.40	295.8	**71.1**
239	4.5	257	3.84	316.1	**75.6**
255	5.0	285	4.27	337.8	**80.5**
268	5.5	309	4.65	355.4	**84.3**
279	6.0	344	5.21	370.4	**87.4**
288	6.5	374	5.69	382.6	**89.8**
298	7.0	401	6.11	396.2	**92.6**
310	7.5	432	6.61	412.4	**95.8**
312	8.0	462	7.10	415.1	**96.0**
318	8.5	487	7.50	423.2	**97.4**
324	9.0	518	8.00	431.3	**98.7**
328	9.5	545	8.44	436.8	**99.5**
331	10.0	575	8.93	440.8	**99.9**
335	10.5	606	9.43	446.2	**100.6**
338	11.0	632	9.85	450.3	**101.0**
339	11.5	660	10.31	451.6	**100.8**
339	12.0	689	10.79	451.6	**100.2**
340	12.5	719	11.28	453.0	**100.0**
340	13.0	749	11.78	453.0	**99.4**

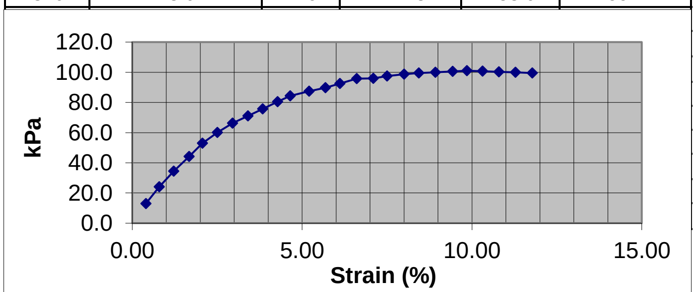

Photo of Failed Speciman

Max. Compressive Stress (kPa)
101.0

Pocket Pen kPa	Torvane kPa
Top	Top
75	30
50	30
50	30
75	27
63 Avg	**29** Avg
Bottom	Bottom
50	30
75	28
75	28
50	30
63 Avg	**29** Avg
	1 Torvane Factor

Reporting of these test results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.

Amec Foster Wheeler Environment & infrastructure

Per.

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Manager; Technical Services

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Rev 2014a

UNCONFINED COMPRESSION

Figure B4

IN ACCORDANCE WITH ASTM D2166

TO: **MMM Group**

OFFICE: Winnipeg
PROJECT NO: **WX17932**
COPIES TO:
TEST DATE: **May 25, 2016**

ATTENTION:

PROJECT: **Ruby & Aubrey Outfall Gate Chambers**

TEST HOLE: **2** **SAMPLE #:** 20
TECHNICIAN: **VM** **DEPTH:** 35-37ft

	Yes	No
ATTERBERG LIMITS:		**X**
HYDROMETER:		**X**

WET WEIGHT: 1137.7 g
MOISTURE (%): 33.3

SAMPLE DIAMETER: 70.10 **mm**, 70.30 **mm**, 71.12
AVERAGE: 70.51 **mm**

INITIAL AREA: 3904.36 **mm²**
LENGTH: 145.57 **mm**, 145.55 **mm**
AVERAGE: 145.56 **mm**
VOLUME: 0.000568 **m³**

DENSITY
WET: **2001.9** (kg/m³)
DRY: **1501.8** (kg/m³)

STRAIN RATE: 0.99 **%/min.** (to failure) (0.5% to 2% per min as per ASTM)

Load Dial (0.0001")	Elapsed Time (min)	Strain Dial (0.001")	Strain %	Axial Load (2014 Data)	Stress Load (kPa)
34	0.5	32	0.50	48.6	**12.4**
74	1.0	60	0.92	106.7	**27.1**
106	1.5	90	1.39	147.2	**37.2**
127	2.0	120	1.87	171.5	**43.1**
140	2.5	148	2.34	186.5	**46.7**
151	3.0	178	2.84	199.4	**49.6**
159	3.5	207	3.33	209.9	**52.0**
165	4.0	235	3.81	217.7	**53.6**
170	4.5	263	4.29	224.2	**55.0**
173	5.0	292	4.79	228.1	**55.6**
177	5.5	320	5.28	233.4	**56.6**
179	6.0	350	5.80	236.0	**56.9**
181	6.5	380	6.32	238.6	**57.2**
182	7.0	410	6.84	239.9	**57.2**
183	7.5	440	7.36	241.2	**57.2**
183	8.0	470	7.88	241.2	**56.9**
182	8.5	500	8.41	239.9	**56.3**

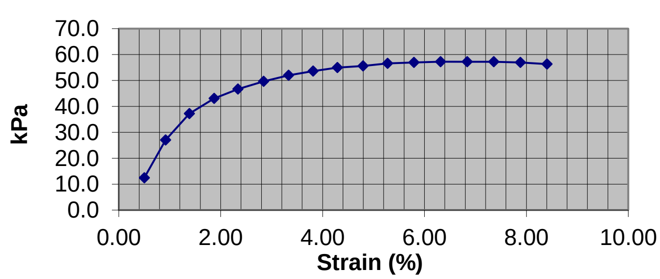

Photo of Failed Speciman

Max. Compressive Stress (kPa)
57.2

Pocket Pen kPa	Torvane kPa
Top	Top
20	30
35	20
25	25
20	30
25 Avg	**26** Avg
Bottom	Bottom
125	60
150	55
120	60
125	62
130 Avg	**59** Avg
	1 Torvane Factor

Reporting of these test results constitutes a testing service only.
Engineering interpretation or evaluation of the test results is
provided only on written request.

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