Winnipeg

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Cockburn and Calrossie Combined Sewer Relief Works C4 – 2700 Trunk Sewer Geotechnical Baseline Report

FINAL – REV 2

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1.0 INTRODUCTION

1.1 GENERAL

The City of Winnipeg is completing a combined sewer relief project for the Cockburn and Calrossie districts including the construction of a Land Drainage System (LDS) trunk sewer pipe from the proposed Parker Storm Retention Basin (SRB) to Taylor Ave. installed by trenchless construction methods.

1.2 PURPOSE OF REPORT AND LIMITATIONS

This Geotechnical Baseline Report (GBR) summarizes the geotechnical condition observed along the alignment from the proposed Parker SRB to Taylor Ave. and provides construction considerations that will form part of the basis of the design for the Work and is intended for use by bidders as an aid in bid preparation. This report includes:

- Description of the project;
- Interpretations of the geologic and geotechnical data collected from the project;
- Summary of anticipated subsurface conditions along the alignment;
- Key design considerations for the various components of the project; and
- A discussion of some of the important construction considerations that the Contractor will need to address during bid preparation and construction.

The results of the geotechnical investigation carried out at the proposed site are presented in the latest revision of KGS Group's final report titled Cockburn and Calrossie Combined Sewer Relief Works C4 – 2700 Trunk Sewer Geotechnical Data Report.

This report presents the geotechnical engineer's best judgement of the subsurface and ground conditions anticipated to be encountered at the project site during construction. The soil stratigraphy has been interpolated between the test holes that were drilled along the alignment.
To facilitate the project, certain assumptions were made with respect to the construction methods and the level of workmanship that can reasonably be expected for this project. It

should be noted that the Contractor's selected equipment, means, methods, and workmanship will influence the behaviour of the subsurface soils at the site.

The geotechnical data related to the subsurface conditions contained within this report are intended for the exclusive use of the City of Winnipeg, their Consultants and the Contractor, if necessary, in evaluating the merits of differing site condition claims that may arise during construction. Some of the technical concepts, terminologies, and descriptions in this report may not be fully understood by bidders. The Contract Documents require that the bidders confer with a qualified geotechnical engineer or engineering geologist who is familiar with all aspects of this report and the Geotechnical Data Report. This engineer should have experience in subsurface conditions similar to those described herein, and should carefully review and explain this information so that a complete understanding of the information presented can be developed prior to submitting a bid.

The Geotechnical Baseline Report has been prepared in accordance with the guidelines and practices described in the Geotechnical Baseline Reports for Construction, Suggested Guidelines, published by ASCE, 2007.

Certain elements of the work are based on set requirements, including but not limited to the following:

- Nominal pipe diameter and
- Alignment and invert elevation of the proposed LDS.

Elements of the project which are flexible for the Contractor in the means and method, subjected to approval by the City of Winnipeg include but are not limited to the following:

- Trenchless construction technique and
- Location, size and number of launching and retrieval shafts.

2.0 PROJECT DESCRIPTION

2.1 GENERAL

The description and dimensions for the various components of the project provided in this report are approximate and for illustration purposes only. The Contractor should refer to the Contract Drawings for precise information on the dimensions and project layout.

2.2 PROJECT LOCATION

The project site is located in Winnipeg, Manitoba. The alignment runs between Manitoba Hydro's and Shindico's property on a City of Winnipeg Right of Way as shown on the Contract Drawings. The trunk sewer will extend from Taylor Ave to the proposed outlet south of the CN crossing at the proposed Parker SRB. The location of the trunk sewer is shown on the Construction Drawings.

2.3 WINNIPEG CLIMATE

Winnipeg is located in central southern Manitoba at the bottom of the Red River Valley, a low-lying flood plain with flat topography. Winnipeg has a humid continental climate with a wide range of temperatures throughout the year. The monthly average temperature ranges from -18°C in January to 20°C in July. Winter is defined as the time which the daily mean temperature remains below 0°C and typically lasts from the beginning of November to the beginning of April. Spring and autumn are defined as the time period the mean daily temperature ranges from 0° to 6°C and are typically short in duration, lasting only a couple of weeks.

The average yearly precipitation in Winnipeg is 505 mm of precipitation per year although the precipitation can vary greatly. The average annual snow fall in Winnipeg is 115 cm, with the most snow typically accumulating in January and February.

2.4 KEY COMPONENTS OF THE PROJECT

The LDS trunk sewer pipe will be 2700 mm in diameter, approximately 510 m in length and will convey water across Taylor and Parker Lands from Taylor Ave to the proposed Parker SRB. The invert elevations of the truck sewer are shown on the Construction Drawings. Based on the alignment and length of the pipe, a minimum of three (3) shaft locations are anticipated to be required for the installation of the trenchless sewer pipe.

3.0 SOURCE OF INFORMATION

The following references were referred to in the preparation of this GBR.

3.1 GEOTECHNICAL INVESTIGATIONS

1. KGS Group, “Cockburn and Calrossie Combined Sewer Relief Works C4 – 2700 Trunk Sewer Geotechnical Data Report, October 2016 ”.

3.2 GEOTECHNICAL GUIDELINES AND STANDARDS

1. American Society of Civil Engineers, 2007, “Geotechnical Baseline Reports for Construction, Suggested Guidelines”, Essex R. J.

2. Canadian Geotechnical Society, 2006 “Canadian Foundation Engineering Manual”, 4th Edition.

3. City of Winnipeg, Standard Construction Specifications, 2016.

3.3 PUBLICATIONS

1. Bannatyne, B. B., 1975, High Calcium Limestone Deposits of Manitoba, Manitoba Mines Branch Publications 75-1.

2. Broms, B.B., Bennemark, H., 1967. Stability of clay at vertical openings. ASCE, Journal of Soil Mechanics and Foundation Engineering Division, SMI 93, 71–94.

3. Department of Geological Engineering, University of Manitoba, (1983). Geological Engineering Report for Urban Development of Winnipeg.

4. Heuer, R.E., 1974. Important Ground Parameters in Soft Ground Tunneling. Proceedings of Subsurface Exploration for Underground Excavation and Heavy Construction. ASCE. New York. 41–55.

5. Hollmann, F., Thewes, M., 2013. Assessment method for clay clogging and disintegration of fines in mechanised tunnelling. TUST 37, 96–106.

6. Peck, R.B., 1969. Deep excavations and tunneling in soft ground. In: 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City State-of-the-Art volume, pp. 225–290.

7. Thewes M. and Burger W. (2004) Clogging risks for TBM drives in clay Tunnels & Tunnelling International, pp.28-31. June.

4.0 GEOLOGICAL SETTING

This Section of the report contains regional geology, general site and subsurface conditions including soil, rock and groundwater along the proposed alignment.

4.1 REGIONAL GEOLOGY

The regional geology of the site has been outlined in the Geotechnical Data Report. Additional information on Winnipeg geology is included in the following references:

- Baracos, A., Shields, D.H., and Kjartanson, B. 1983. Geological engineering report for urban development of Winnipeg. University of Manitoba.
- Baracos, A., Graham, J., Kjartanson, B., and Shields, D.H. 1983. Geology and soil properties of Winnipeg. In ASCE Conference on Geologic Environment and Soil Properties, Houston Tx: 39-56.
- Baracos, A. 1977. Compositional and structural anisotropy of Winnipeg soils – a study based on scanning electron microscopy and X-ray diffraction analyses, Canadian Geotechnical Journal, 14: 125-137.
- Baracos, A., Graham, J., and Domaschuk, L. 1980. Yielding and rupture in a lacustrine clay, Canadian Geotechnical Journal, 17: 559-573.
- Quigley, R.M. 1968. Soil mineralogy Winnipeg swelling clays. Can. Geotech. J. 5(2), pp. 120-122.
- Render, F.W. 1970. Geohydrology of the metropolitan Winnipeg area as related to groundwater supply and construction, Canadian Geotechnical Journal, 7(3): 243-274.
- Skaftfeld, K. (2014). Experience *as a Guide to Geotechnical Practice in Winnipeg* (Masters of Science Thesis). University of Manitoba, Winnipeg, Manitoba.

4.2 SOURCES OF GEOLOGIC AND GEOTECHNICAL INFORMATION

Geological data for the project site is available from several sources, including the Geotechnical Data Report, and published maps and reports. A compilation of the available information and data including results of the geotechnical drilling and laboratory test data obtained from the 2015 and 2016 investigations are presented in the GDR.

The information contained within the GDR takes precedence over those from previous geotechnical reports that have been compiled for other components of the Cockburn and Calrossie Combined Sewer Relief Works project.

4.3 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were performed in 2015 and 2016 as part of Contract 3 and 4 for the Cockburn and Calrossie Combined Sewer Relief Works. The 2015 investigation consisted of drilling five (5) test holes at the proposed Parker SRB site.

An additional geotechnical investigation was performed in 2016 as part of Contract 4. The 2016 investigation included drilling nine (9) test holes along the proposed trunk sewer alignment.

One (1) standpipe piezometer and twelve (12) pneumatic piezometers were installed in the 2015 and 2016 test holes, to monitor groundwater levels in the clay, till and bedrock.

Laboratory testing was performed on representative soil samples obtained from both of these investigations. Details of the field and laboratory programs, as well as the geotechnical data obtained from these investigations, are presented in the Geotechnical Data Report.

4.4 GROUNDWATER CONDITIONS

Groundwater level measurements obtained from the standpipe and pneumatic piezometers within the proposed project alignment are summarized on the Table 1. These measurements indicate that groundwater will be encountered during the excavation of the shafts and tunneling.

5.0 PREVIOUS TUNNEL CONSTRUCTION EXPERIENCE

5.1 GENERAL

Trenchless pipe installation using Microtunnel Boring Machine (MTBM) particularly for pipe diameter greater than 2500 mm is not common in Winnipeg and there is limited local experience with trenchless installation of such large diameter (2700 mm) pipes.

5.2 SOUTHWEST RAPID TRANSIT CORRIDOR LAND DRAINAGE SEWER INSTALLATION

The Southwest Rapid Transit Corridor Land Drainage Sewer project was completed by the City of Winnipeg as a Design Bid Build Project. The work included the installation of approximately 600 m of LDS pipe with a Tunnel Boring Machine (TBM). The work was completed south of the Winnipeg Transit bus garage, from Osborne Street to the Rapid Transit Corridor. The work was completed approximately 2 km northwest of the proposed Cockburn/Calrossie trunk sewer.

The project was tendered as the installation of a 1350 mm diameter LDS installed with trenchless methods. The contract was awarded to Nelson River Construction, using their Akkerman Tunnel Boring Machine.

The concrete LDS was installed approximately 7.5 m below grade in the clay deposit. The installation included 3 runs, approximately 200 m in length each. Launch sites were approximately 12 m x 5 m x 8 m deep. The shoring for the launching shafts was designed by a professional engineer and consisted of soldier piles (steel) with timber lagging. Receiving shafts were circular, approximately 3 m in diameter and 7 m deep.

Each run was completed within 30 mm of the designed vertical grade and no issues were encountered with the selected trenchless installation method.

No particular constructability issue was encountered during the installation of the pipe. The followings lessons can be taken from previous work completed in Winnipeg:

- A detailed geotechnical investigation should be completed to determine the soil stratigraphy and relevant engineering properties along the alignment of the pipe, and
- Groundwater levels should be monitored to determine the potential for seepage inflow and basal heave during excavations for the shafts.

6.0 GROUND CHARACTERIZATION

The general stratigraphy for the project site has been developed based on the information obtained from the 2015 and 2016 exploratory test holes, laboratory test data and our extensive experience with the local geology. The stratigraphy and engineering properties of the overburden soil deposits and bedrock unit are presented in this Section. Detailed test hole log records and results of laboratory tests are provided in the Geotechnical Data Report.

6.1 OVERBURDEN CHARACTERIZATION

The stratigraphy at the site consisted of a shallow surficial layer of organic clay overlaying a thin silt deposit. Beneath the silt deposit is an extensive layer of high plastic clay overlying dense glacial silt till deposit and limestone bedrock (see Figure 1 for simplified stratigraphic profile). Varying thicknesses of fill was observed in five (5) test holes, TH16-01, TH16-03, TH16-04, TH16-06 and TH16-07 ranging from organic clay fill to silty clay fill and sand and gravel fill.

The overburden stratigraphy has been divided into five (5) layers, as follows:

- Organic clay.
- Fill.
- Silts.
- Clay.
- Glacial till.

The division of the soil layers is based on visual classification in the field and laboratory testing.

6.1.1 Organic Clay

Organic clay was encountered in four (4) of the test holes, TH16-05, TH16-06, TH16-07 and TH16-08, extending to a maximum depth of 2.3 m below existing grade. The organic clay was mottled brown-black to black in colour, damp to moist, firm to stiff in consistency, of intermediate plasticity and contained trace to some sand and gravel. The undrained shear strength, as estimated from the field Torvane, ranged from 75 to 90 kPa in the organic clay.

6.1.2 Fill

A layer of fill was encountered in the five (5) test holes, TH16-01, TH16-03, TH16-04, TH16-06 and TH16-07. The fill material ranged from sand and gravel fill to silty clay fill and organic clay fill. The fill thickness ranged from 0.9 m to 2.1 m.

The sand and gravel fill encountered in test holes TH16-01 and TH16-07 was brown in colour, dry to wet, loose to compact, well graded and contained some fine to coarse grained gravel.

The silty clay fill observed in test hole TH16-06 was mottled brown and black in colour, damp, stiff in consistency, of high plasticity and contained some fine to coarse grained sand and trace organics. The average undrained shear strength, as estimated from the field Torvane was 75 kPa.

Organic clay fill was encountered in two (2) test holes, TH16-03 and TH16-04 ranging in thickness from 0.9 to 2.1 m± thick. The organic clay fill was black in colour, damp to moist, firm to stiff, of intermediate plasticity and contained trace to some fine to coarse grained sand and gravel. The undrained shear strength, as estimated from the field Torvane, ranged from 45 to 90 kPa in the organic clay.

6.1.3 Silts

A silt to silty clay layer approximately 0.3 to 1.5 m± thick was encountered in all of the test holes at depths ranging from 0.6 to 3.0 m± below the ground surface. The silt layer was tan to brown in colour, moist to wet, soft in consistency, of low plasticity and contained some sand. Typical moisture contents for silt deposits are indicated in the GDR. Seepage is commonly observed within this silt layer.

6.1.4 Clay

An extensive layer of highly plastic clay was encountered at elevations ranging from approximately 230.2 to 232.4 m±. The thickness of the high plastic clay ranged from 10.4 to 13.1 m. This deposit will be encountered during the excavation for the shafts and along the proposed alignment for the tunnel. The upper layer of the clay deposit was mottled brown in colour and extended to approximately elevation 224.7 to 227.8 m±. The upper clay deposit was damp to moist, of high plasticity and stiff in consistency. The consistency decreases with depth from stiff to firm. The lower clay deposit was grey, moist to wet, of high plasticity, and soft to firm in consistency, becoming softer with depth.

The clay deposit contained some silt lenses/pockets and trace to some fine to coarse grained sand. These non-plastic, non-clay material generally occur throughout the clay deposit as varves, veins, seams, inclusions or pockets that are typically less than a centimeter in diameter. The tendency for horizontal orientation of the varves, veins, and seams introduce a visible macrostructure to the clay and are a contributing cause for the observed anisotropy in horizontal permeability and strength of the deposit. Quigley (1968) offers the explanation that frozen silt lumps were rafted into glacial Lake Agassiz by icebergs and dropped into the clays as frozen lumps. Baracos (1977) provided a more likely explanation, considering the sharply defined boundaries of the inclusions, that they were deposited not frozen but as cemented or lithified material which subsequently disintegrated into silt. The high contents of dolomite and calcite (determined from X-ray diffraction by Baracos 1977) suggest that the fragments were of limestone or similar origin.

The undrained shear strength, as estimated from the field Torvane, ranged from 25 to 90 kPa with an average of 55 kPa in the upper clay and 18 to 55 kPa with an average of 30 kPa in the lower clay. Figure 2 shows variation of undrained shear strength in clay deposit with elevation.

Unconfined compressive strength testing was completed on clay samples taken within the proposed trunk sewer alignment. The measured unconfined compressive strength ranged from 43 to 117 kPa with an average of 64 kPa. There is good correlation between the undrained shear strength estimated from the field Trovane and the unconfined compressive strength.

Liquid and Plastic Limits, Plastic Indices, moisture contents and unconfined compressive strengths are outlined in the GDR and summarized on Table 2. The majority of the laboratory testing results from the 2016 investigation for the clay deposit is within the typical ranges for the Winnipeg area.

6.1.5 Glacial Till

Silt till deposit was encountered below the clay deposit at elevations ranging from 219.3 and 220.8 m±. The excavation of the shaft or shoring may extend into the dense silt till deposit. A layer of clay till was encountered in test hole TH16-09 below the silt till at an elevation of 218.7 m±. The silt till deposit ranged in thickness from 0.6 to 2.8 m±. The silt till was found to be tan in colour, damp, dense to very dense, of low plasticity, and contained some fine to coarse grained sand and gravel. Boulders and cobbles are commonly found within the till layer and should be anticipated within the deposit at the project site.

The Standard Penetration Test (SPT) blow counts for 300 mm ranged from 22 to greater than 50 blows. The till was classified as very dense (greater than 50 blows for 300 mm) for seven (7) of the SPTs. A summary of the uncorrected SPT N values recorded in the silt till are presented in Table 3 of this report.

6.1.6 Boulders

Cobbles and boulders were not directly observed during the geotechnical investigation. Premature refusal of SPT spoons in the test holes within the till deposit typically indicate the presence of cobbles and boulders in the silt till or at the bedrock surface. The pipe will be installed within the clay layer, approximately 7 m above the silt till interface and the tunneling should not be impacted by the cobbles and boulders within the silt till, however, the shoring for the shaft construction may require penetration into the dense till.

6.2 BEDROCK

The bedrock encountered at the proposed site consisted of limestone. The depth to limestone bedrock varied from 13.7 to 14.3 m±, below grade at elevations ranging from 218.7 to 219.4 m±. The bedrock will not be encountered during the tunneling or excavation of the launching/retrieving shafts.

6.2.1 Rock Quality Designation

The Rock Quality Designation (RQD) ranged from 19% to 83% classifying the rock as poor to good. Table 4 shows the distribution of the RQD within the cored test holes.

6.3 GROUNDWATER CONDITIONS

Ten (10) pneumatic piezometers were installed in the test holes during the 2016 geotechnical investigation. Five (5) piezometers were installed in the clay layer, two (2) piezometers were installed in the silt till layer and three (3) piezometers were installed in the bedrock. After installation the piezometers were read four (4) times.

In general a slight downward gradient from the clay into the silt till and bedrock was observed from the most of the groundwater monitoring data. Typically the groundwater reading in the clay ranged from elevation 224.3 to 230.6 m, from elevations 226.3 to 227.4 m in the silt till and from elevation 224.7 to 227.1 m in the bedrock. Details of the piezometer installation and groundwater readings are outlined in the Geotechnical Data Report.

Groundwater levels fluctuate seasonally and typically rise during the spring melt and after significant rainfall events and/or snowmelts.

For baseline purposes, the groundwater elevation within the various strata is presented on Table 5. The groundwater levels in Table 5 are approximately 1.0 m higher than the readings recorded during the groundwater monitoring period. The additional 1.0 m adjustment is to account for potential seasonal fluctuations.

TABLE 5
BASELINE GROUNDWATER LEVELS

Soil Strata	Groundwater Elevation (m)
Clay	231.6
Till	228.4
Bedrock	228.1

6.4 BASELINE VALUES

- For baseline purposes, the undrained shear strength of the clay deposit varies uniformly with depth from 70 kPa at El. 233 m to 20 kPa at El. 225 m. The undrained shear strength of the clay is 20 kPa below El. 225 m. These strengths are representative of clay at its natural moisture content. The measured undrained shear strengths and baseline undrained shear strengths are shown on Figure 2.
- The baseline effective shear strength parameters and permeability of each soil strata are outlined in Table 6.
- For baseline purposes, the average Liquid and Plastic limits of the clay are 92% and 25%, respectively. The variation of Plasticity Index with elevation is shown on Figure 3.

TABLE 6
BASELINE EFFECTIVE SHEAR STRENGTH PARAMETERS

Material Type	c' (kPa)	Φ' (degrees)	γ (kN/m^3)	K_{sat} (m/sec)
Silts	5.0	14	18.5	$1x10^{-6}$
Upper Brown Clay	5.0	14	18.5	$1x10^{-8}$
Lower Grey Clay	5.0	14	18	$1x10^{-10}$
Till	5.0	23	22	$1x10^{-6}$

6.4.1 Swelling Potential of Clay Deposit

The swelling potential of a clay soil can be categorized based on the plasticity and percentage of clay sized particles (Figure 15.5, Canadian Foundation Engineering Manual, 4th Edition). The swelling potential of clay is highest when a sample has a high percentage of clay size particles and a high plasticity index. Clay minerals accounts for between 67 and 81 percent of the total composition of the lake Agassiz Clay in Winnipeg. The clays size fractions generally consist of approximately 75 percent montmorillonite, 10 percent illite and 10 percent kaolinite and approximately 5 % quartz mineral. Over-consolidation ratio of the clay is generally less than 2.

The clay at the site is classified to have a very high potential severity of an expansive soil based on the laboratory testing completed and is subject to considerable volume change with change in moisture content. Volumetric increases are usually in the 2% range with swelling pressure generally less than 75 kPa. For baseline purposes, it should be assumed that the clay layer present at the site has very high swelling potential. The variability of moisture content in the clay deposit with elevation is shown on Figure 4.

7.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

7.1 TRENCHLESS PIPE INSTALLATION METHODS

The installation may be completed with an appropriately designed Tunnel Boring Machine (TBM) or microtunnel boring machine subject to the detailed requirements of the technical specifications. The trenchless installation method selected must be compatible with geological condition outlined in this Baseline Report and must take into account the size of pipe, space limitation at the site and other constraints that have been identified in the Contract Documents.

Design and construction consideration for the trenchless pipe installation methods are provided in this section.

7.2 LAUNCHING AND RECEIVING SHAFTS

A minimum of three (3) shaft locations are anticipated to be required for the installation of the trenchless sewer pipe as shown on the Construction Drawings. The shafts will be constructed primarily within the clay deposit and may extend into the underlying till. General design and construction considerations are outlined below:

- The shaft locations will be used to launch and/or receive the trenchless tunneling equipment and provide space for construction activities.
- The shaft will be excavated through the clay and shoring may penetrate into the silt till. The pipe inverts are shown on the Construction Drawings.
- The Contractor is responsible for the design of the shoring and temporary support systems at the shaft locations.
- The temporary support systems must be designed to resist lateral earth pressures, lateral hydrostatic pressures, surcharge of equipment/material stockpiled adjacent to the shaft and control ground movements in accordance with the Contract documents.
- Baseline groundwater levels are outlined on Table 5. A base slab capable of resisting buoyance and basal heave is required at the shaft locations unless the Contractor can demonstrate that heave is not a concern and that pressures can be relieved in a controlled manner.

- The design of the shaft complies with Manitoba Workplace Safety and Health Act and Regulation. The Contractor shall be required to submit design details and drawings for their shafts to the City of Winnipeg for approval.
- All seepage water pumped from the shaft locations will be discharged according to the requirements outlined in the Contract Documents.

7.2.1 Base Heave

The base of excavation and shoring should be designed to achieve a minimum factor of safety of 1.5 with respect to basal heave. Installation of groundwater monitoring well will be required at the location of each shaft to measure the piezometric elevation in the vicinity of the shaft during construction. Depending on the groundwater conditions at the time of construction, groundwater depressurization may be required to achieve the specified factor of safety against basal heave.

7.2.2 Care and Control of Water

In order to maintain safe working conditions in the excavation and to protect against instability of the excavation base, water will not be allowed to accumulate anywhere within the excavations. Effective drainage and sump pump systems will be required below the base of excavation to maintain a firm, dry working surface. The Contractor shall design the internal drainage system to efficiently collect groundwater seepage and all water inflow draining into the excavation shall be pumped out and treated or use a watertight concrete slab designed to resist uplift. The Contractor will be responsible to prevent surface runoff from entering the excavation.

7.3 TUNNELS

General design and construction considerations for the tunnels are outlined below:

- The Contract Documents indicate the trunk sewer will be installed by a trenchless construction method.
- The Contractor is required to determine the trenchless construction method and design the necessary components.
- The Contractor must pick suitable equipment to properly handle the excavated material.
- The trunk sewer will be installed within the clay layer, approximately 7 m above the glacial silt till deposit.

- The properties of the clay soil are outlined in Section 6 and within the GDR. The clay layer has a very high swelling potential, mitigation measures such as increasing the size of the overcut may be required depending on the trenchless pipe installation method selected by the Contractor. Furthermore, any activity that may result in a drastic change in the moisture content of the cay (drying or wetting) may aggravate the potential for swelling and should be avoided.
- The Contractor is required to collect and discharge groundwater flows according to the Construction Documents.

7.3.1 Tunnel Face Stability

The ground behavior during tunnel excavation through the clay deposit can be expected to be "squeezing". Descriptions of the terms "squeezing" are based on the Tunnelman's Classification System. This classification system was developed for classifying tunneling conditions in soil. For baseline purposes, it is the classification system adopted for this GBR. Descriptions of these terms are provided in Appendix A.

Broms and Bennemark, (1967) identified the overload factor, N, as the fundamental ratio for characterizing the instability of the face. The overload factor for tunneling though cohesive soils is defined as:

$$N = \frac{\gamma H}{S_U}$$

where H is the depth to tunnel axis, γ is the soil unit weight, and S_U is the undrained shear strength of the ground prior to excavation. Field observations (Peck, 1969) show that N values ranging from 5 to 7 typically result in tunneling difficulties and may cause tunnel face instability.

Variation of the estimated overload factor through the tunnel axis is shown in Figure 5. The overload factor varied from 2 near the crown to up to 6 near the invert. The estimated overload factor indicates that an unsupported tunnel face through the clay formation would have a moderate squeezing potential below El 226 m.

7.3.2 Stickiness Potential and Clogging Risks

The clay deposit present at the site has a tendency to develop sticky behavior (adhesion of clay material to each other or to metal surface). This stickiness may result in the clogging and blockage of the cutterhead, work chamber, screw conveyors (EPBM -Earth Pressure Balance Machines), slurry lines or prevent the shield advancement due to excessive friction.

The potential for clogging while tunneling through the clay formation was evaluated using the chart suggested by Hollmann and Thewes (2013). Atterberg limits (Liquid limit, Plastic limit, and natural moisture content) of clay samples tested in the Laboratory and their Plasticity Indices were plotted on Figure 6 to determine the corresponding clogging potential of the clay. It should be noted that the Hollmann and Thewes chart (Figure 6) was developed from data collected from fluid supported shield drives but are assumed to be applicable to other tunneling methods. For baseline purposes, the clay deposit at the site has high stickiness and strong clogging potential.

The additional effort that will be required for cleaning clogged components may lead to significant reduction in productivity and increased cost. Therefore, the tunneling system and separation plant (for slurry supported shield drive) used for this project should be designed to mitigate this potential clogging problem. Thewes and Burger (2004) suggested the following upgrades to the design of the TBM to mitigate the risk of clogging:

- Enlarge passages and other obstructions in the transport of clay chips from the tunnel face to the slurry line.
- Increase the ratio of suspension flow rate to the volume of excavated soil to prevent accumulation of clay in the chamber (circuit and flushing concepts).
- Avoid clay agglomeration by increasing agitation in areas prone to material settlement
- Avoid Closed-type cutting wheel.

Other mitigating measures include optimizing the cutting tools penetration to get a favorable clay chip size, the use chemical additives and provision of high pressure water jets in the cutter head chamber.

7.4 TEMPORARY EXCAVATIONS

Temporary excavations will be required to facilitate the construction of the proposed trunk sewer. All excavation work are required to be performed in accordance with the Workplace Safety and Health Act and Part 26 of the Manitoba Workplace Safety and Health Regulation, M.R. 217/2006.

Baseline groundwater levels are presented on Table 5. Baseline soil strengths for temporary excavation design are outlined in Table 6.

Excavations performed adjacent to the existing roadway or infrastructure, require temporary shoring or bracing. Excavations deeper than 1.5 m are required to be designed and approved prior to construction by an experienced professional engineer with an expertise in geotechnical engineering. The shoring design should account for all applicable surcharge loads. Opening and voids behind shoring lagging or sheet piles will be backfilled with free draining granular materials.

The silt layers are known to be water bearing and are susceptible to strength loss when subjected to mechanical disturbance and sloughing from wetting. All open excavation side slopes will be covered with water proof material to prevent saturation of the soil and all surface runoff will be directed away from the excavations. The Contractor will maintain all surcharge loads such as stockpiled soil, equipment, etc. a minimum of 10 m away from the edge of excavations.

During the site investigations water infiltration was observed in some of the test holes as discussed in the GDR.

7.5 IMPACT ON EXISTING STRUCTURES

Excavation support systems will be designed by the Contractor to control ground movement/subsidence around the perimeter of the excavation. Potential settlement of the ground surface adjacent to temporary shoring system should be recognized and accounted for in the design. Any resulting movement/settlement around the perimeter of the excavation and of utilities, roadways, railways and buildings must be kept within acceptable limit as specified in the contract document. The Contractor will maintain specified clearances from buried utilities and infrastructure as indicated in the Construction Documents.

The Contractor should be experienced to avoid improper use of the trenchless installation equipment resulting in additional settlement.

The excavation and shoring system will be designed by a professional engineer with extensive relevant experienced and the works must be inspected and certified by the same professional engineer to verify that the temporary structure has been installed according to the design.

7.5.1 Considerations for Pipe Installation at CN Rail Crossing

Construction of the section of the pipe installation that will be installed beneath the existing CN Rail and within the CN right of way line shall be completed using a trenchless method and must comply with the following specification and standards:

1. A guide to the Pipe and Wire Process- water/Sewer Pipeline by CN Rail (August 2009).
2. Pipeline Crossing Specifications by CN Rail.
3. Transport Canada standard, “TC E-10 Standards Respecting Pipeline Crossings under Railways”.
4. Safety Guidelines for Contractors and Non-CN Personnel, May 2004.

7.6 INSTRUMENTATION PROGRAM

The Contractor is required to monitor the potential impact of the pipe installation on adjacent structures, including commercial/retail buildings. Instrumentation is required to be installed to monitor ground movements, settlement of any structures within the zone of influence, ground vibration and noise levels. The threshold values and amount of displacement allowed is outlined within the Contractor Documents.

7.7 GROUNDWATER MANAGEMENT AND SPOIL DISPOSAL

The Contractor is expected to be familiar with and follow all local spoil disposal regulations including all monitoring, analysis, permits and treatment required by the City of Winnipeg. Transportation and disposal of the spoil material is required to comply with all applicable laws and regulations and be in accordance with the Contract Documents. Discharge of groundwater must following requirements outlined in the Contract Documents and the Contractor is required to obtain all necessary permits/approvals. Routine monitoring of groundwater discharge quality by the Contractor will be required during construction.

7.8 FROST PENETRATION

The expected depth of frost penetration has been estimated assuming a design freezing index of 2680^0C days, taken as the coldest winter over a ten (10) year period. The estimated maximum depth of frost penetration is 2.5 m assuming no insulation cover.

7.9 CORROSION POTENTIAL

The degree of exposure of concrete in contact with soils to sulphate attack is classified in CAN/CSAA23.1-M94 (Concrete Materials and Methods of Concrete Construction) as moderate (S-3), severe (S-2) or very severe (S-1). All concrete utilized in foundation elements should have a minimum specified 28 day compressive strength of 35 MPa and class of exposure of S-1, corresponding to very severe sulphate attack. A maximum water to cement ratio of 0.40 should be specified in accordance with Table 2, CSA A23.1-09 for concrete with very severe sulphate exposure (S-1). Concrete which may be exposed to freezing and thawing should be adequately air entrained to improve freeze-thaw durability in accordance with Table 4, CSA A23.1-09.

8.0 STATEMENT OF LIMITATIONS

8.1 THIRD PARTY USE OF REPORT

This report has been prepared for the City of Winnipeg, their Consultants, and the Contractor and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions undertaken based on this report.

8.2 GEOTECHNICAL INVESTIGATION STATEMENT OF LIMITATIONS

The geotechnical investigation findings and recommendations of this report were prepared in accordance with generally accepted professional engineering principles and practice. The findings and recommendations are based on the results of field and laboratory investigations, combined with an interpolation of soil and groundwater conditions found at and within the depth of the test holes drilled by KGS Group at this site. If conditions encountered during construction appear to be different from those shown by the test holes drilled by KGS Group or if the assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be reviewed and modified if necessary.

TABLES

TABLE 1
GROUNDWATER MEASUREMENTS

Test Hole:	**TH15-05**	**TH16-05 (I9)**		**TH16-06 (SHAFT A)**		**TH16-07 (I3)**		**TH16-08 (SHAFT B)**		**TH16-09 (SHAFT C)**	
Ground Elevation (m):	**232.80**	**233.15**	**233.15**	**233.27**	**233.27**	**233.99**	**233.99**	**233.30**	**233.30**	**232.73**	**232.73**
Piezometer No.:	**SP**	**36898**	**36890**	**36895**	**36891**	**36894**	**36892**	**36896**	**36893**	**36897**	**36889**
Tip Elevation (m):	**218.58**	**224.62**	**218.52**	**225.95**	**218.03**	**225.15**	**218.45**	**225.98**	**218.36**	**224.2**	**218.1**
Monitoring Zone:	**Silt till**	**Clay**	**Bedrock**	**Clay**	**Silt Till**	**Clay**	**Silt Till**	**Clay**	**Bedrock**	**Clay**	**Bedrock**
Date		**Piezometric Elevation (m)**									
7-Jul-15	225.08		-	-	-	-	-	-	-	-	-
14-Oct-15	225.25		-	-	-	-	-	-	-	-	-
25-May-16		230.03	226.36	227.47	226.30	229.97	(Note 1)	230.57	225.22	226.42	225.72
17-Jun-16	225.60	229.60	226.40	227.47	226.68	230.05	(Note 1)	230.50	224.86	226.42	225.65
26-Aug-16	225.17	229.52	227.10	227.47	227.39	229.90	(Note 1)	230.57	224.65	224.32	224.86
6-Oct-16	-	229.60	227.10	227.90	227.39	229.60	(Note 1)	230.36	224.07	225.62	225.36

Note 1: Erroneous reading

It should be noted that groundwater levels will fluctuate seasonally and following precipitation events.

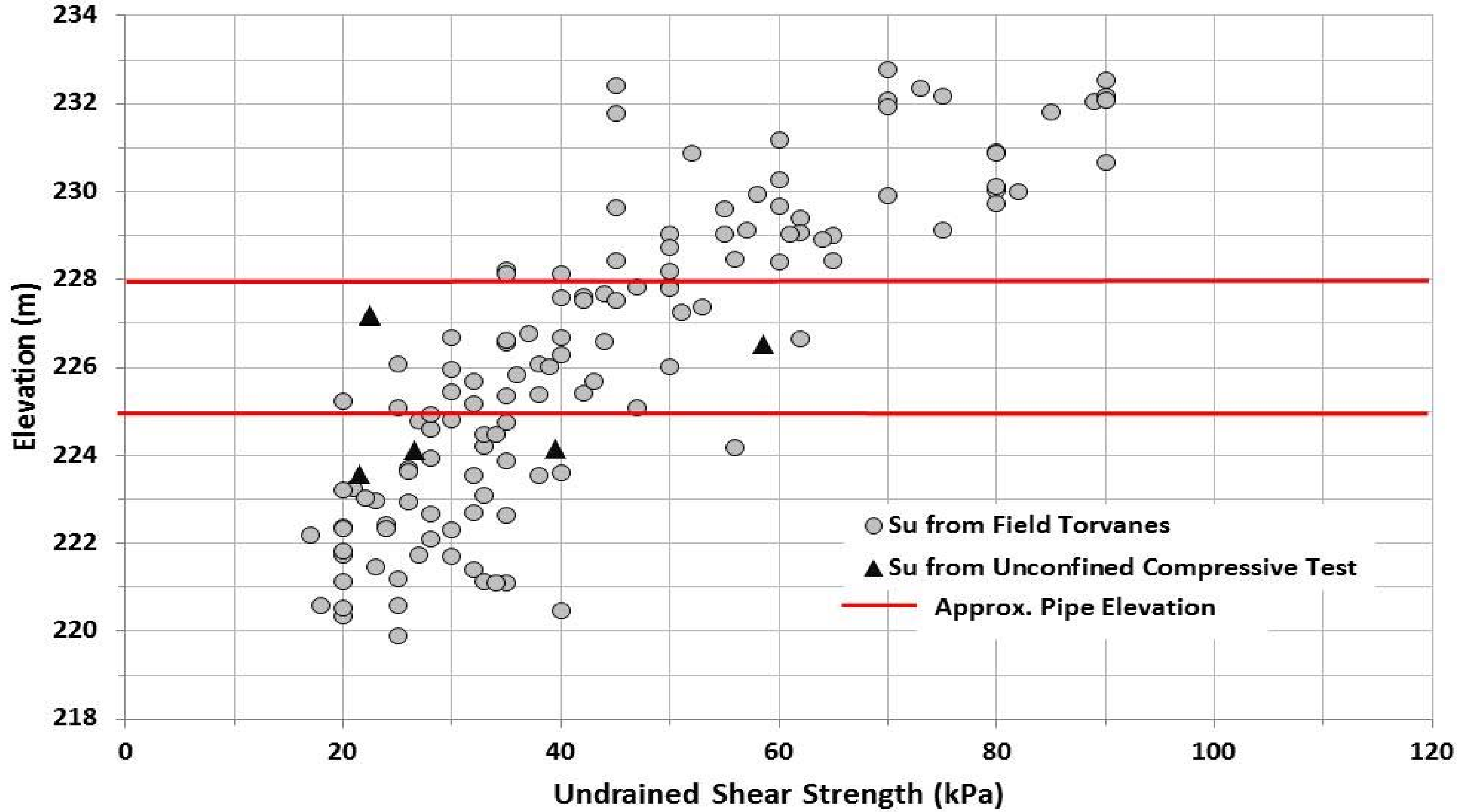
TABLE 3
GLACIAL TILL – SPT SUMMARY

Density	Frequency
Very Loose (0-4 blows/0.3 m)	
Loose (4-10 blows/0.3 m)	
Compact (10-30 blows/0.3 m)	4
Dense (30-50 blows/0.3 m)	
Very Dense (greater than 50 blows/0.3 m)	1
Spoon Refusal (greater than 50 blows for less than 0.3 m)	6

TABLE 4
LIMESTONE BEDROCK – RQD SUMMARY

Rock Quality Designation	Frequency
Very poor (0-25%)	
Poor (26-50%)	1
Fair (51-75%)	
Good (76-90%)	2
Excellent (91-100%)	

FIGURES



NOTES:

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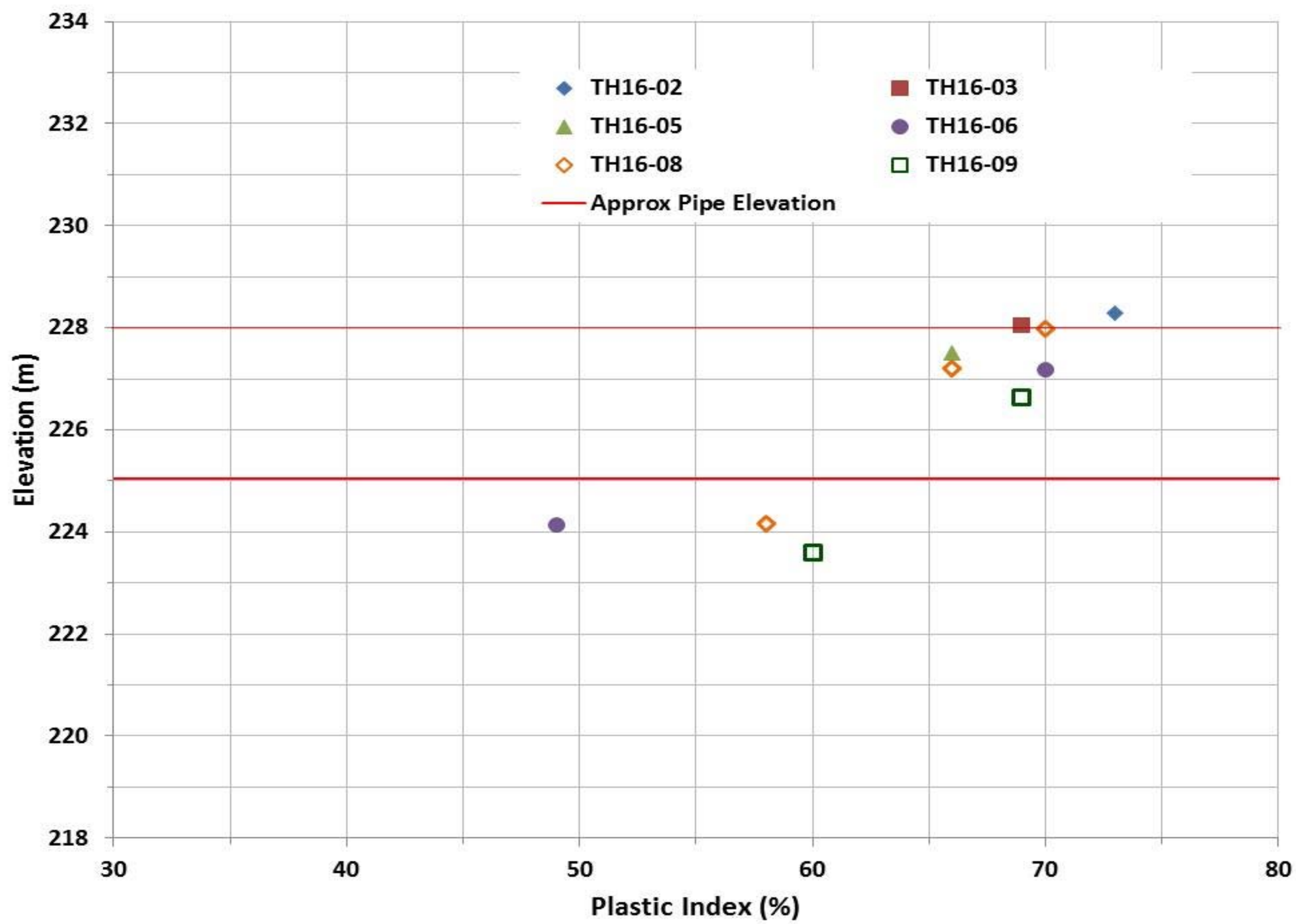
Winnipeg

Cockburn/ Calrossie Combined Sewer Relief Project

Geotechnical Baseline Report

Undrained Shear Strength of Clay Deposit

September 2016	Figure 2	1 Rev



NOTES:

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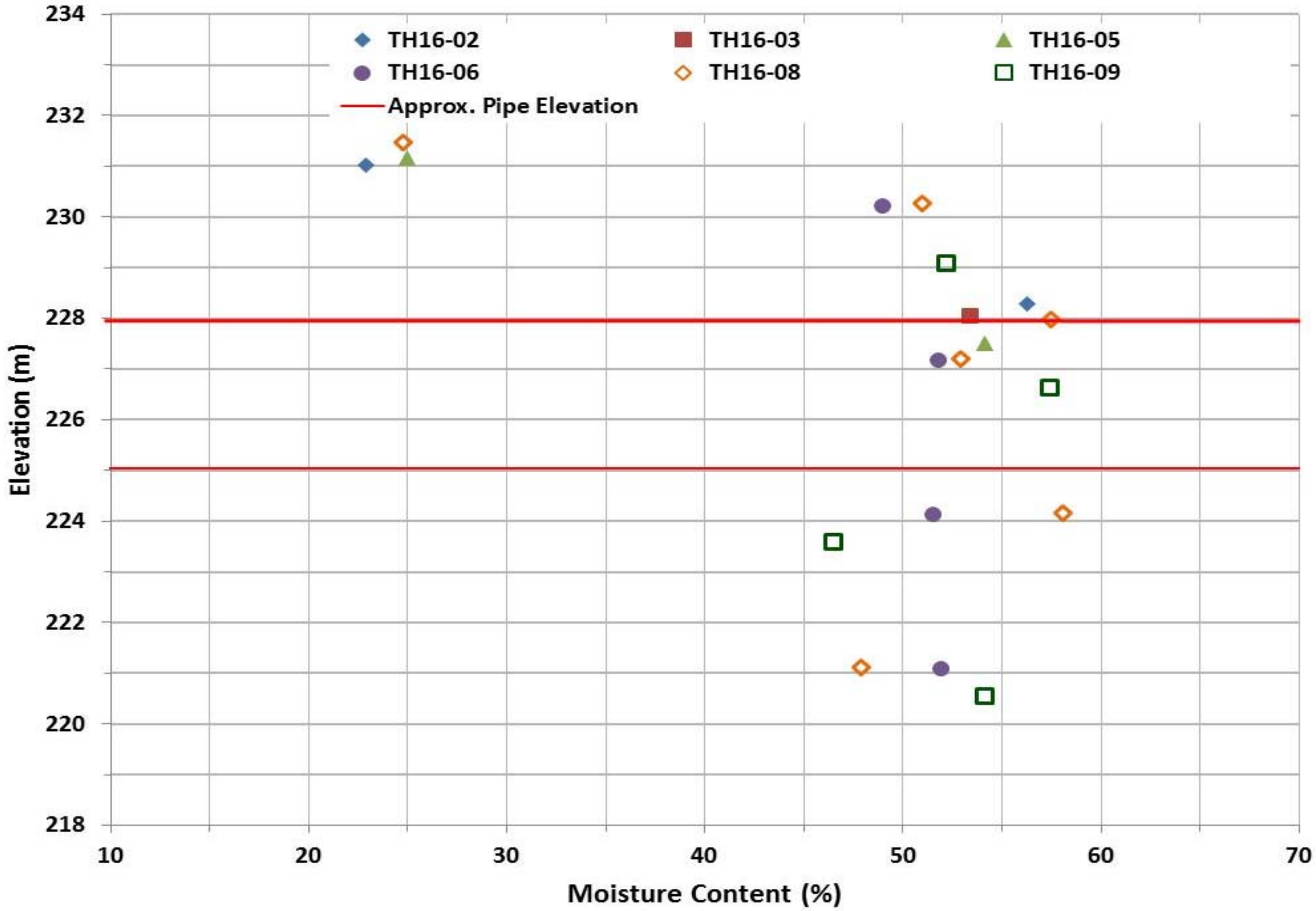
Geotechnical Baseline Report

Plasticity Index of Clay versus Elevation

September 2016	Figure 3	1 Rev



NOTES:

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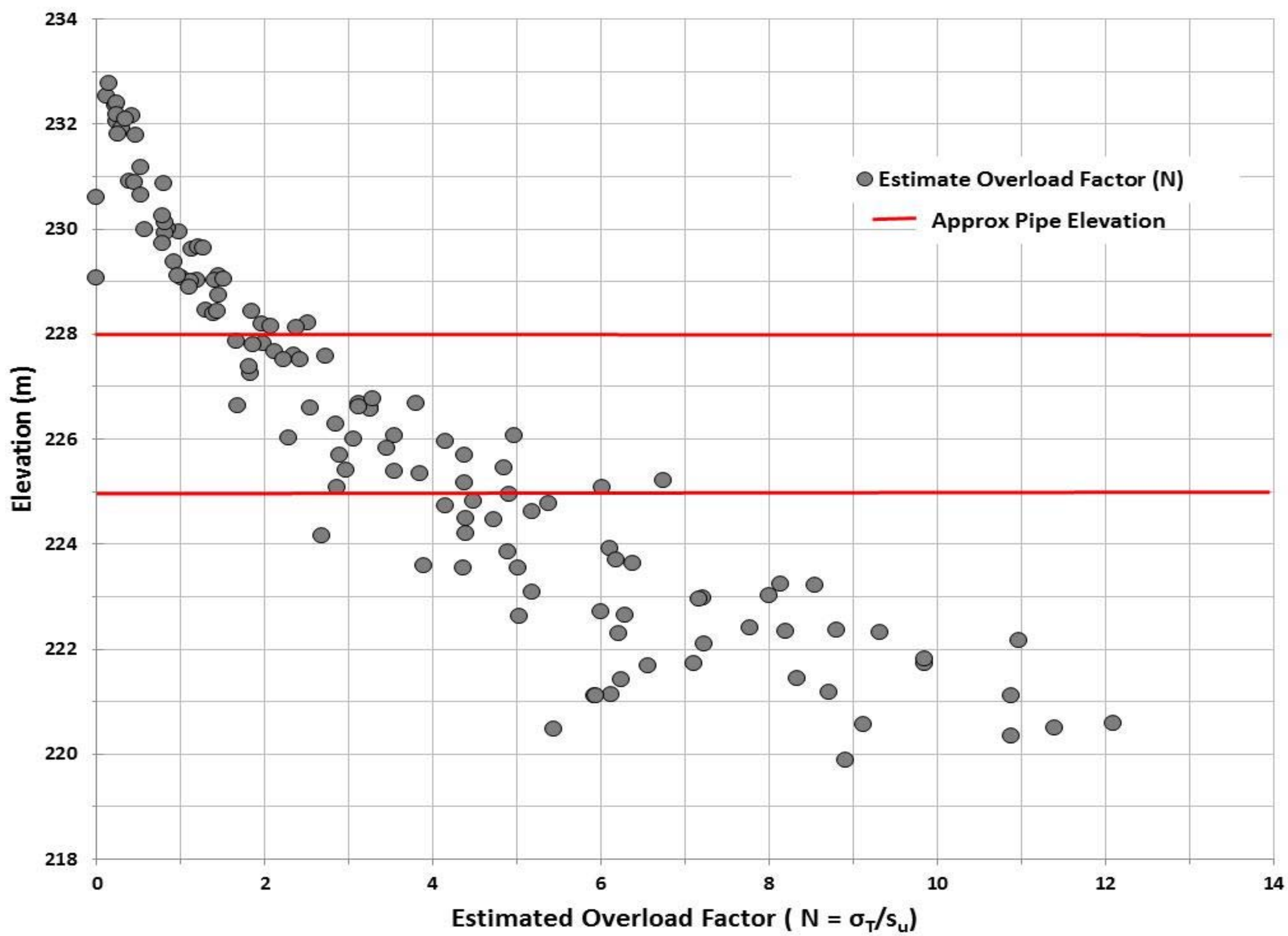
Geotechnical Baseline Report

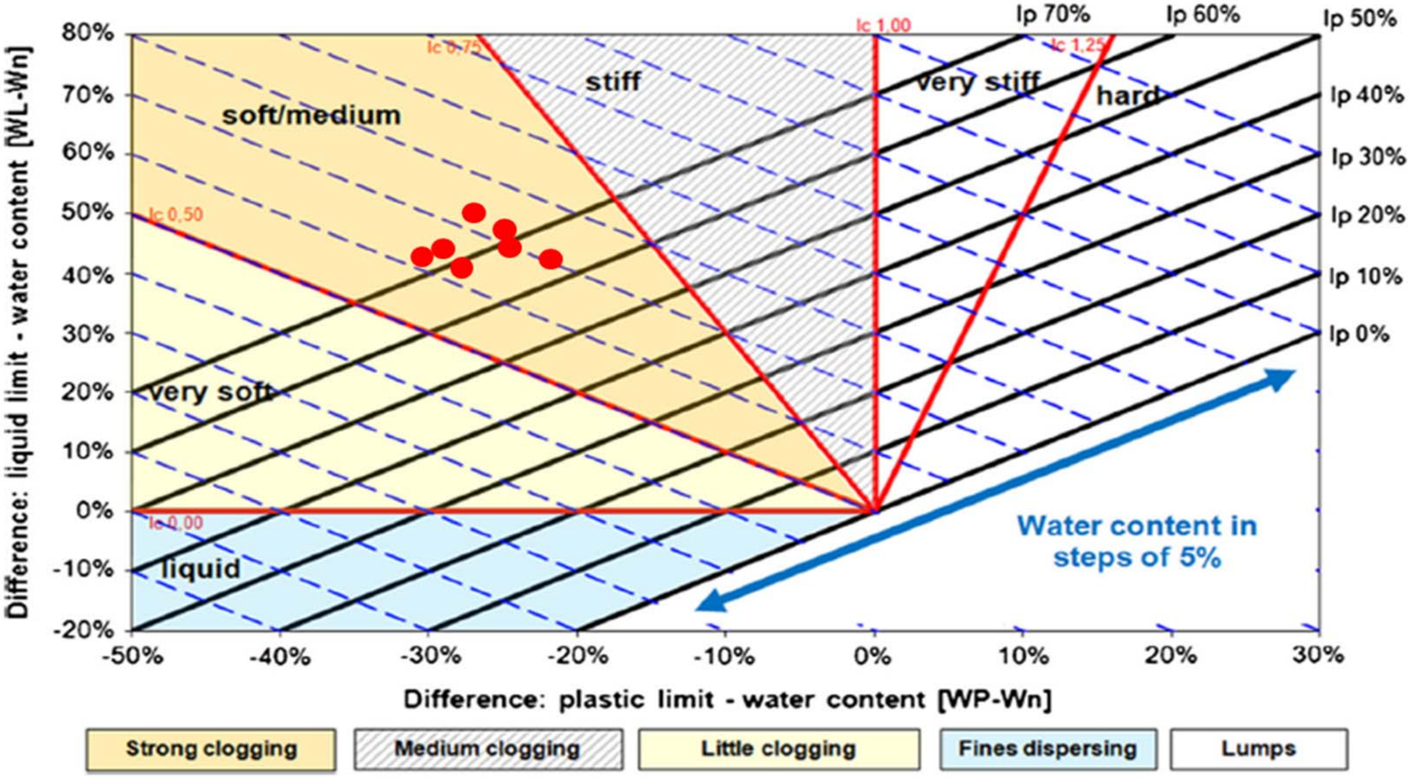
Moisture Content versus Elevation

September 2016	Figure 4	1 Rev



NOTES:

KGS GROUP	Winnipeg	
Cockburn/ Calrossie Combined Sewer Relief Project		
Geotechnical Baseline Report		
Estimated Overload Factor (N) in Clay versus Elevation		
September 2016	Figure 5	1 Rev



NOTES:
Adapted from Hollmann and Thewes (2013)

KGS GROUP	Winnipeg	
Cockburn/ Calrossie Combined Sewer Relief Project		
Geotechnical Baseline Report		
Clogging Potential of the Clay at the Tunnel Location		
September 2016	Figure 6	1 Rev

APPENDIX A

TUNNELMAN'S GROUND CLASSIFICATION

Tunnelman's Ground Classification for Soils[1]

Classification		Behavior	Typical Soil Types
Firm		Heading can be advanced without initial support, and final lining can be constructed before ground starts to move.	Loess above water table; hard clay, marl, cemented sand and gravel when not highly overstressed.
Raveling	Slow raveling ----- Fast raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to over-stress and "brittle" fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.	Residual soils or sand with small amounts of binder may be fast raveling below the water tale, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.
Squeezing		Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.	Ground with low frictional strength. Rate of squeeze depends on degree of overstress. Occurs at shallow to medium depth in clay of very soft to medium consistency. Stiff to hard clay under high cover may move in combination of raveling at excavation surface and squeezing at depth behind surface.
Running	Cohesive - running ----- Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (+/- 30° – 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.	Clean, dry granular materials. Apparent cohesion in moist sand, or weak cementation in any granular soil, may allow the material to stand for a brief period of raveling before it breaks down and runs. Such behavior is cohesive-running.
Flowing		A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown, and walls, and can flow for great distances, completely filling the tunnel in some cases.	Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.
Swelling		Ground absorbs water, increases in volume, and expands slowly into the tunnel.	Highly preconsolidated clay with plasticity index in excess of about 30, generally containing significant percentages of montmorillonite.

1 Modified by Heuer (1974) from Terzaghi (1950)