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Cockburn and Calrossie Combined Sewer Relief Works C5 –Taylor Ave Trunk Sewer Geotechnical Baseline Report

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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 along Taylor Ave. from Wentworth to Nathaniel Street. The proposed LDS pipe project is part of the Cockburn/Calrossie Combined Sewer Relief Works currently being undertaken by the City of Winnipeg.

The LDS trunk sewer will consist of 2400 mm pipe from Wentworth Street to Wilton Street. The LDS sewer will drain to the 2700 mm LDS sewer recently installed (Contract 4) along Wilton Street. Open-face rotary wheel Tunnel Boring Machine (TBM) two-pass tunnelling or pipe jacking will be employed for the installation of the proposed pipe.

1.2 PURPOSE OF REPORT AND LIMITATIONS

This Geotechnical Baseline Report (GBR) summarizes the geotechnical condition observed from Wentworth St. to Wilton St. along the Taylor Ave. alignment and provides construction considerations that 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 encountered 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 Geotechnical Data Report (GDR) ("Cockburn and Calrossie Combined Sewer Relief Works C5 – Taylor Ave Trunk Sewer Geotechnical Data Report – Final – Rev 1", KGS, 2019)

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. 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;
- Pipe materials;
- Use of one launch and one reception shaft; AND
- Construction of an intermediate manhole using pre-installed excavation support to minimize occupancy of Taylor Road.

Alignment and invert elevation of the proposed LDSElements of the project which are flexible for the Contractor in the means and method, subject to approval by the City of Winnipeg include but are not limited to the following:

- Auger boring method to install stub connections;
- Shaft drilling method to install tee riser manholes;
- 2-pass or pipe jacking methods to install the LDS;
- Ground support system for 2-pass tunneling;
- TBM tunnelling method; and
- Size and shoring system for 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 LDS sewer runs from Wentworth St. to Wilton St. along Taylor Ave., as shown on the Contract 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 sewer pipe will be 2400 mm in diameter from Wentworth St. to Wilton St., approximately 700 m in length. The LDS sewer pipe will tie into the existing 2700 mm sewer pipe constructed using the microtunnelling method along Wilton St. south of Taylor Ave. and draining towards the proposed Parker Storm Retention Basin. The invert elevations of the LDS sewer are shown on the Construction Drawings.

A 2100 stub will be installed in Taylor Ave. to the west of Wilton for a future connection.

3.0 SOURCE OF INFORMATION

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

3.1 GEOTECHNICAL INVESTIGATIONS

1. KGS Group, January 2019, "Cockburn and Calrossie Combined Sewer Relief Works C5 – Taylor Ave Trunk Sewer Geotechnical Data Report – Final – Rev 1".

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

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, 2017, Standard Construction Specifications.

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 Tunnelling. 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 tunnelling 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. June 2004, Clogging risks for TBM drives in clay Tunnels & Tunnelling International, pp.28-31.

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:

1. Baracos, A., Shields, D.H., and Kjartanson, B., 1983, Geological engineering report for urban development of Winnipeg. University of Manitoba.
2. 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.
3. 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.
4. Baracos, A., Graham, J., and Domaschuk, L., 1980, Yielding and rupture in a lacustrine clay, Canadian Geotechnical Journal, 17: 559-573.
5. Quigley, R.M., 1968, Soil mineralogy Winnipeg swelling clays. Can. Geotech. J. 5(2), pp. 120-122.
6. Render, F.W., 1970, Geohydrology of the metropolitan Winnipeg area as related to groundwater supply and construction, Canadian Geotechnical Journal, 7(3): 243-274.
7. 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 (GDR), 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 2016 and 2017 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 2016 and 2017 as part of Contracts 4 and 5 for the Cockburn and Calrossie Combined Sewer Relief Works. The 2016 Contract 4 investigation consisted of drilling nine (9) test holes for the Wilton St. LDS trunk sewer from Taylor Ave. to the proposed Parker Storm Retention Basin. Two (2) pneumatic piezometers were installed within the Contract 5 alignment during the 2016 geotechnical investigation. The 2017 investigation consisted of drilling seventeen (17) test holes along the proposed trunk sewer alignment. Nine (9) standpipe piezometers were installed within the silt till and bedrock and six (6) vibrating wire piezometers were installed within the clay, to monitor groundwater levels.

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, vibrating wire 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 tunnelling.

5.0 PREVIOUS TUNNEL CONSTRUCTION EXPERIENCE

5.1 GENERAL

Trenchless pipe installation using a TBM, particularly for pipe diameter greater than 2400 mm is not common in Winnipeg and there is limited local experience with trenchless installation of larger diameter pipes. Microtunnelling was used for Contracts 2 and 4 for the Cockburn and Calrossie Sewer Relief Works. Details of this work and lessons learnt are outlined in the following Sections.

5.2 CONSTRUCTION OF TRUNK SEWER AND LDS SEPERATION – CONTRACT TWO - COCKBURN AND CALROSSIE COMBINED SEWER RELIEF PROJECT

Contract 2 of the Cockburn and Calrossie Combined Sewer Relief Project included the approximately 1,300 m of 1200 mm diameter by trenchless installation methods. All the LDS pipes were installed within the brown clay layer approximately 8 to 9 m below grade.

The work was completed on Byng Place, Rockman Street, Parker Ave and Heatherdale Ave approximately 600m south from the Taylor Ave alignment. The work was awarded to Marathon Drilling Co, using a Herrenknecht AVN1200 microtunnel boring machine (MTBM).

Issues encountered during the shaft construction and pipe installation include the transfer of vibrations through the clay layer and efficient separation of the clay particles within the separation plant.

The Contractor’s shaft design consisted of sheet piling and walers. The sheet piling was vibrated through the clay layer to the design elevation approximately 12 m below grade. Due to the massive nature of the clay this process resulted in the transfer of the vibrations to nearby structures causing potential damage.

During the tunnelling process, the separation plant was not effectively separating the water from the excavated clay. The original separation plant produced mud spoil too wet to be hauled offsite without the addition of a drier material. Marathon Drilling Co. modified the separation plant to optimize the return water. Modifications that were not successful included replacing the screens with coarse ones which resulted in an excess of excavated material entering the recovery tank and increasing the chute size on the shaker deck to increase the area for the material to spread. Adding sprayer bars to force material through the screens was moderately successful and adding a scalping belt, with adjustable belt speed, was the most successful method they used.

The followings lessons can be taken from previous work completed in Winnipeg:

- Proper separation plants designed for clay soils are required; and
- Vibrating loading does not quickly dissipate within the clay layer and can cause structural damage to adjacent structures. Alternative installation methods should be explored for the installation sheet piling, if required for the shaft locations.

5.3 CONSTRUCTION OF TRUNK SEWER AND LDS SEPERATION – CONTRACT FOUR - COCKBURN AND CALROSSIE COMBINED SEWER RELIEF PROJECT

Contract 4 of the Cockburn and Calrossie Combined Sewer Relief Project included the installation of approximately 525 m of 2700 mm diameter LDS by microtunnelling methods. All the LDS pipes were installed within the brown and grey clay layers approximately 8 to 8.5 m below grade.

The work was completed adjacent to Manitoba Hydro and Shindico property along Wilton St. from south side of the CN crossing within the Parker Lands to the north side of Taylor Ave The work was awarded to Ward & Burke Microtunnelling Ltd., using a Herrenknecht AVN 2500 MTBM. The MTBM was up-skinned to match the outside diameter of the 2700 mm reinforced concrete jacking pipe. The Contract 5 tunnelled LDS will intersect the Contract 4 LDS at Taylor Ave and Wilton St.

No significant issues were encountered during the microtunnelling operations. During the shaft excavation and tunnelling process no large boulders were observed. The contractor used dual centrifuges to effectively remove the clay from the slurry, and the Contractor’s sinking caisson shaft design and installation methodology was highly effective.

During the tunnelling process, a correlation was observed between the face pressure maintained at the MTBM and surface settlements. During the first tunnel drive crossing the CN right-of-way, the machine face pressure was near zero prior to crossing under the railway tracks. Initial surface settlements measured along the centerline alignment as the MTBM passed underneath two sets of surface monitoring points exceeded the 25 mm tolerance value. Tunnelling operations ceased until the Contractor increased the face pressure by pumping bentonite slurry to the machine and tunnel to fully charge the annular overcut. An average face pressure of 55 kPa was maintained for the remaining drive length and the initial settlement values were reduced by approximately half (10-15 mm) as the MTBM passed underneath.

During the second tunnelling drive along Wilton, the average face pressure was gradually decreased from 45 kPa to 15 kPa over the total drive length of 400 m to maintain low jacking forces as the tunnel progressed. The resulting initial surface settlement along the centerline alignment at each set of monitoring points was observed to increase with the decreasing face pressure. Initial settlement at 70 m± of tunnelling was on the order of 16 mm as compared to 21 mm at 360 m± of tunnelling. In all cases the settlement values on both tunnelling drives were observed to incrementally rebound and stabilize as the MTBM travelled further away from each set of monitoring points and the concrete pipe was jacked through the tunnel due to the pressurized bentonite slurry in the annulus.

Ward & Burke utilized a Herrenknecht automated bentonite lubrication system to systematically maintain bentonite pressure in the annulus outside the jacking pipe. This system was highly effective in minimizing the jacking forces, although a component of reduced forces was attributed to the decision to maintain low face pressure during the drives.

Contact grouting of the tunnel annulus was highly effective in restoring the surface to pre-tunnelling elevations. Ward & Burke used a lubrication port spacing of 15 m (every 5 pipes) during tunnelling operations. During contact grouting of the first tunnelling drive under the CN tracks, the bentonite lubrication in the annulus was not viscous enough to be displaced through the subsequent set of lubrication ports. Surface cracks indicating ground heave were observed as grouting pressures were increased while attempting to displace the bentonite slurry. The friction force in the bentonite slurry were too high and may have been the result of the density being too high, the slurry mixing with the in-situ clay particles in the annulus, or the wide spacing of the lubrication ports. As a result, grout bulkheads were created on the north and south sides of the CN crossing to seal and maintain the stabilized annular pressure under each set of railway tracks. The bentonite lubrication density was decreased during the second tunnelling drive and was observed to be displaced during contact grouting at the same 15 m port spacing. Measured surface settlements along the centerline of the second drive rebounded during contact grouting to as little as 2 mm from the baseline values.

The Contractor's shaft design consisted of a cast-in-place, reinforced concrete caisson with sacrificial steel cutting shoe cast-in to the first pour of the caisson wall. The shafts were installed by excavating the soil within the caisson as sinking occurred under the self-weight of the cutting shoe and concrete wall. Each shaft consisted of four or five concrete pours that were formed above grade and sunk to a total depth of 9 to 9.5 m below grade. No additional point or vibratory loads were required when sinking the shafts in the in-situ clay layer. Negligible vibrations were produced during shaft installation and the Contractor's design and methodology were comparatively non-intrusive to the surrounding environments.

The followings lessons can be taken from previous work completed in Winnipeg:

- Settlement will occur as a result of tunnelling. Observed settlement is a function of applied face pressure.
- Lubrication port spacing and bentonite lubrication mix design should be given extra consideration when working in clays with high stickiness potential.
- Contact grouting can be effective in restoring the ground surface elevation to pre-tunnelling conditions if proper lubrication and grout port spacing is selected.
- The concrete caisson shaft design and self-sinking installation methodology produced negligible vibrations through the clay layer and was comparatively non-intrusive to the surrounding environment.

6.0 GROUND CHARACTERIZATION

The general stratigraphy for the project site has been developed based on the information obtained from the 2016 and 2017 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 consists of pavement with granular base overlaying clay with a thin silt deposit at shallow depths. A layer of clay fill was encountered in test hole TH17-15 below the granular fill layer. Test hole TH17-16 consisted of topsoil overlying silt as it was drilled in the park space north west of the Taylor Ave. and Nathaniel St., intersection. Beneath the silt deposit is an extensive layer of highly plastic clay overlying glacial silt and limestone bedrock (see Figure 1 for simplified stratigraphic profile).

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

- Pavement structure;
- Fill;
- Silt;
- Clay; and
- Glacial till.

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

6.1.1 Pavement Structure

Test holes TH17-01 to TH17-15 and TH17-17 were drilled on the road surface and approximately 0.2 to 0.3 m of asphalt and concrete was observed in these test hole. A granular base material was observed below the pavement ranging in thickness from 0.05 to 0.3 m. Water infiltration was observed from the granular layer in test hole TH16-11 where 0.3 m of granular base was observed.

6.1.2 Fill

A layer of fill was encountered in test hole TH17-15. The fill material was approximately 0.5 m thick and consisted of 0.4 m± of clay fill and 0.1 m± of wood. The clay fill was black in colour, damp, stiff in consistency, of high plasticity and contained some organics.

6.1.3 Silt (ML)

A silt layer approximately 0.1 to 1.1 m± thick was encountered in twelve (12) of the test holes at elevations ranging from 229.1 to 232.1 m±. The silt layer was tan in colour, moist, soft in consistency, and of low plasticity. Two (2) silt layers were observed in test hole TH17-04 from elevation 230.1 to 230.7 m± and from 231.1 to 231.4 m±. Seepage is commonly observed within this silt layer.

6.1.4 Clay (CH)

An extensive layer of highly plastic clay was encountered at elevations ranging from approximately 230.8 to 232.2 m±. The thickness of the highly plastic clay ranged from 9.7 to 11.9 m±. This deposit will be encountered during the excavation for the shafts and along the proposed tunnel alignment. The upper layer of the clay deposit was mottled brown in colour and extended to approximately elevation 224.5 to 227.4 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, of high plasticity, and soft to firm in consistency, becoming softer with depth.

The clay deposit contained some silt inclusions and trace to some fine to coarse grained sand. These non-plastic, non-clay materials 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 undrained shear strength, as estimated from the field Torvane, ranged from 35 to 100 kPa with an average of 60 kPa in the upper clay and 20 to 50 kPa with an average of 33 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 48 to 117 kPa with an average of 82 kPa. There is good correlation between the undrained shear strength estimated from the field Torvane 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 2017 investigation for the clay deposit are within the typical ranges for the Winnipeg area.

XRD analysis was completed on four (4) clay samples from within the proposed LDS alignment. The results of the testing indicated the quartz content of the clay samples ranged from 16.1 to 20.2%, the clinochlore content ranged from 13.3 to 17.0%, the muscovite content ranged from 15.4 to 29.3%, the calcite content ranged from 0.6 to 4.5%, the dolomite content ranged from 4.2 to 9.7%, and the smectite content ranged from 28.6 to 37.1%. Laboratory testing results are included in the GDR. High smectite content is often associated with high clogging potential during tunnelling.

6.1.5 Glacial Till (ML)

Silt till deposit was encountered below the clay deposit at elevations ranging from 219.0 and 220.5 m±. The excavation of the shaft or shoring may extend into the dense silt till deposit. The silt till deposit ranged in thickness from 0.6 to 3.7m±. 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 was found to be tan in colour, damp, loose to very dense, of low plasticity, and contained some to with 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 4 to greater than 50 blows. The till was classified as very dense (greater than 50 blows for 300 mm) for thirteen (13) 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. Occasional cobbles and boulders were observed within the clay layer during previous tunnelling projects within the vicinity of this project. The LDS pipe will be installed within the clay layer, approximately 5 m above the silt till interface. The tunnelling should not be impacted by the cobbles and boulders within the silt till; however, occasional cobbles may be encountered in the clay layer.

6.2 BEDROCK

The majority of the bedrock encountered at the site was dolomite with dolomite limestone, limestone and interbedded shale and dolomite observed in some of the core holes. The elevation of bedrock varied from El. 216.6 to 219 m±. The bedrock will not be encountered during the tunnelling or excavation of the launching/retrieving shafts.

6.2.1 Rock Quality Designation

The Rock Quality Designation (RQD) ranged from 0% to 92% classifying the rock as very poor to excellent. Table 4 shows the distribution of the RQD within the cored test holes.

6.3 GROUNDWATER CONDITIONS

Nine (9) standpipe piezometers and six (6) vibrating wire piezometers were installed in the test holes during the 2017 geotechnical investigation. Six (6) vibrating wire piezometers were installed in the clay layer, five (5) standpipe piezometers were installed in the silt till layer and four (4) standpipe piezometers were installed in the bedrock. Two (2) pneumatic piezometers were installed within the Contract 5 work area during the Contract 4 2016 geotechnical investigation.

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. The groundwater reading ranged from elevation 224.32 to 231.37 m± in the clay, from elevations 224.63 to 228.06 m± in the silt till and from elevation 224.52 to 228.12 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	232.4
Till	229.1
Bedrock	229.1

6.4 BASELINE VALUES

- For baseline purposes, the undrained shear strength of the clay deposit varies uniformly with depth from 60 kPa at El. 230 m to 20 kPa at El. 223 m. The undrained shear strength of the clay is 20 kPa below El. 223 m and 60 kPa above El. 230 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 typically consist of up to 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 Contractor is to select between directly installing the LDS pipe using pipe jacking techniques or installing the LDS in excavation support installed using a TBM (two-pass method).

The pipe jacking installation will be completed with an appropriately designed open-face rotary wheel TBM, earth pressure balance TBM (EPBM), or MTBM subject to the detailed requirements of the technical specifications. The jacking pipe will be reinforced concrete pipe. The 2-pass installation will be completed with an appropriately designed open-face rotary wheel TBM subject to the detailed requirements of the technical specifications. An Earth Pressure Balance (EPB) TBM may also be used and operated in “open” or “closed” mode. A digger shield was specifically not identified as an appropriate tunnelling shield for use on this project because of the potential for additional ground loss at the face and the resulting settlement causing damage to the pavement and shallow utilities. The carrier pipe for the two-pass option will be CCFRPM. Excavation support is planned to be steel ribs and timber lagging, although alternative support systems will be considered by the City.

The alignment contains two (2) horizontal curves necessitated by the presence of existing utilities under Taylor Ave. The carrier pipe is planned at a constant 0.13% grade. For the pipe jacking alternative, surveying and guidance systems will be required to negotiate the two (2) resulting compound curves. For the two-pass alternative it is anticipated that the tunnel can be maintained with no slope through the dual horizontal curves to simplify the surveying requirements. The carrier pipe within the tunnel will still be required to maintain a constant vertical slope through the tunnel.

The tunnelling machine must be compatible with the geological condition outlined in this Geotechnical Baseline Report and must take into account the size of pipe, the size of the excavation support, space limitation at the site and other constraints that have been identified in the Contract Documents.

The auger boring method of installing an 1800 steel casing can be used to construct the four (4) stub connections at the Harrow and Stafford intersections. The method is one approach to avoid the need to excavate shafts within Taylor Ave. to make the stub connections, as no "retrieval of a shield" is required. The steel casing is not shown in the Contract Documents and is not required if another trenchless method is used to make the stub connections. In no case shall a method be used that requires the excavation of a pit or shaft within Taylor Ave. to facilitate the stub construction and in no case will auger boring be allowed without steel casing.

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

7.2 LAUNCHING AND RECEIVING SHAFTS

A minimum of one (1) driving shaft, one (1) receiving shaft and one (1) shaft for construction of an intermediate manhole (tee riser manhole) are anticipated to be excavated for the installation of the LDS as shown on the Construction Drawings. The tee riser manhole is planned to be installed using a technique that minimizes disruption to traffic in Taylor Ave. The Contract Documents indicate that for the 2-pass method, a shaft is to be drilled, partially cased, partially backfilled, covered, and traffic reinstated prior to tunnelling. Once the tunnel has been completed and the carrier pipe loaded into the tunnel beyond the affected manhole the Contractor will reoccupy Taylor Ave to install a riser pipe and associated manhole within the shaft. The specific means and methods of tee riser manhole construction are up to the Contractor provided they result in a similar period of street occupancy at each manhole location to the staged approach presented in the Contract Documents.

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 TBM 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; and
- 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 1.3 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 TUNNELLING

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

- The Contract Documents indicate the trunk sewer will be installed by a 2-pass tunnelling technique using an open-face rotary wheel TBM or a pipe jacking technique using an open-face rotary wheel TBM or MTBM;
- The Contractor must pick suitable equipment to properly handle the excavated material;
- The tunnel will be installed within the clay layer, approximately 5 to 6.5 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 clay (drying or wetting) may aggravate the potential for swelling and should be avoided; and
- The Contractor is required to collect and discharge groundwater flows according to the Construction Documents.
- Design of concrete and any reinforcement used in softeyes at the launch and receiving shafts and design of the CLSM used for backfill of any conflicting utilities and the tee riser manholes must be compatible with the selected TBM and in particular the TBM cutterhead arrangement and tooling, which in turn must be compatible with the expected tunnelling conditions.

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 tunnelling 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 tunnelling 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 tunnelling 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 0.5 near the crown to up to 4.5 near the invert. The estimated overload factor indicates that an unsupported tunnel face through the clay formation may have a moderate squeezing potential near the tunnel invert.

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 a metal surface). This stickiness may result in the clogging and blockage of the cutterhead, tooling, work chamber, screw conveyors (EPBM), muck carts, conveyors, slurry lines or prevent the shield advancement due to excessive friction.

The potential for clogging while tunnelling 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 tunnelling 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 tunnelling system and separation plant (for a 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; and
- Avoid closed-type cutting wheels.

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 3.0 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 cement grout.

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, and buildings must be kept within acceptable limit as specified in the contract document. This is of utmost importance at the intersection of Wentworth St. and Taylor Ave. 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.

7.6 INSTRUMENTATION PROGRAM

The Contractor is required to monitor the potential impact of the tunnelling on adjacent structures, including commercial/retail buildings and residential. Instrumentation is required to be installed to monitor ground movements, and settlement of any structures within the zone of influence, ground vibration and noise levels. The threshold values and amount of displacement allowed are 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^{0}C 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:	TH16-09		TH17-01		TH17-06		TH17-07	TH17-10			TH17-12		TH17-13	TH17-15		TH16-16	
Ground Elevation (m):	232.73		232.19	232.19	232.14	232.14	232.18	231.76	231.76	231.76	231.98	231.98	232.10	231.79	231.79	232.14	232.14
Piezometer No.:	36897	36889	VW 1700051	Standpipe	VW 1700053	Standpipe	Standpipe	VW 1700050	Standpipe	Standpipe	VW 1700049	Standpipe	Standpipe	VW 1700048	Standpipe	VW 1702738	Standpipe
Tip Elevation (m):	224.2	218.1	225.48	219.39	226.04	215.38	219.07	224.14	218.50	213.62	226.49	216.28	217.47	224.17	213.81	223.90	216.98
Monitoring Zone:	Clay	Bedrock	Clay	Silt Till	Clay	Bedrock	Silt Till	Clay	Silt Till	Bedrock	Clay	Bedrock	Silt Till	Clay	Bedrock	Clay	Silt Till
Date	Piezometric Elevation (m)																
25-May-16	226.42	225.72															
17-Jun-16	226.42	225.65															
26-Aug-16	224.32	224.86															
6-Oct-16	225.62	225.36															
9-May-17	227.42	227.22	230.92	228.06	230.27	226.66	226.64	230.31	225.19	226.57	228.78	226.43	226.48	231.37	228.12		
14-Jun-2017	227.42	225.79	230.81	225.99	229.98	225.52	225.56	229.99	225.66	225.46	228.65	225.21	225.87	230.92	225.67		
25-Sep-2017	227.40	225.29	230.85	225.38	229.98	224.64	224.63	229.73	225.14	224.65	228.96	224.52	227.75	230.86	225.21		
16-Oct-2017																226.79	225.22
6-Jun-18			-	-	-	-	-	228.96	225.21	224.92	228.94	225.71	227.44	230.87	(See note 2)	227.31	224.99
7-Sep-18	228.18	225.22	-	-	-	-	-	229.10	(See note 1)	224.07	-	-	227.44	-	-	-	-
1-Oct-18			230.78	225.07	230.60	227.82	(See note 2)	-	-	-	229.23	(See note 2)	-	230.87	(See note 2	227.49	224.637
15-Oct-18			-	-	-	-	224.68		(see note 3)		-	-	227.10	-	-	-	-
18-Oct-18			-	-	-	-	-	229.09	(see note 3)	224.70	-	-	-	-	-	-	-

Notes.
1. Erroneous reading.
2. Standpipe filled with water.
3. Standpipe filled with water at time of reading, water in the standpipe was bailed out on October 15, 2018, three (3) days later on October 18, 2018 standpipe was filled with water.

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

TABLE 2
SUMMARY OF 2017 LABORATORY TESTING

Testhole ID	Sample	Sample Depth (m)	Description	Moisture content (%)	Unconfined Compressive Strength (kPa)	Grain Size Analyses						Atterberg Limit		
						Gravel (> 4.75 mm)	Coarse Sand (2-4.75 mm)	Medium Sand (0.425-2 mm)	Fine Sand (0.075-0.425 mm)	Silt (0.002-0.075 mm)	Clay (<0.002 mm)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index
TH17-01	S1	0.6 to 0.9	Clay	29.4										
	S3	3.7 to 4.0	Clay	53.1								102	29	73
	S4	4.6 to 5.2	Clay	55.4	117							110	29	81
	S8	7.6 to 8.2	Clay	52.9	48									
	S12	12.8 to 13.0	Silt till	22.2										
TH17-06	S1	0.9 to 1.2	Clay	34.9										
	S3	3.7 to 4.0	Clay	48.7								98	28	70
	S4	4.6 to 5.2	Clay	53.3	102									
	S6	6.1 to 6.7	Clay	50.0	71							92	24	68
	S9	8.8 to 9.1	Clay	48.4								93	27	66
	S11	11.3 to 11.6	Clay	47.5										
	S13	12.8 to 13.1	Silt till	20.0		45.2	4.7	4.4	7.4	21.4	16.9			
TH17-09	S10	12.8 to 13.1	Silt till	22.8										
	S11	13.4 to 13.7	Silt till	13.1		9.4	6.2	10	14.9	39.2	20.3			
TH17-10	S1	0.9 to 1.2	Clay	34.2										
	S3	3.7 to 4.0	Clay	54.2								106	31	75
	S6	6.1 to 6.7	Clay	47.8	66							86	24	62
	S9	8.2 to 8.5	Clay	53.0										
	S11	11.3 to 11.6	Clay	48.5								88	24	64
	S12	13.1 to 13.4	Silt till	14.8		3.5	6.4	11.3	14.8	46.9	17.1			
TH17-12	S4	4.6 to 5.2	Clay	54.0	81									
	S8	7.6 to 8.2	Clay	50.9	84									
TH17-13	S9	12.8 to 13.1	Silt till	16.0										
	S11	14.3 to 14.6	Silt till	12.7		46.5	17.7	14.2	8.4	9.8	3.4			
TH17-15	S1	1.2 to 1.5	Clay	36.0										
	S4	4.6 to 5.2	Clay	51.8	88							108	28	80
	S6	6.1 to 6.7	Clay	50.4	82							91	26	65
	S12	12.8 to 13.1	Silt till	13.0										

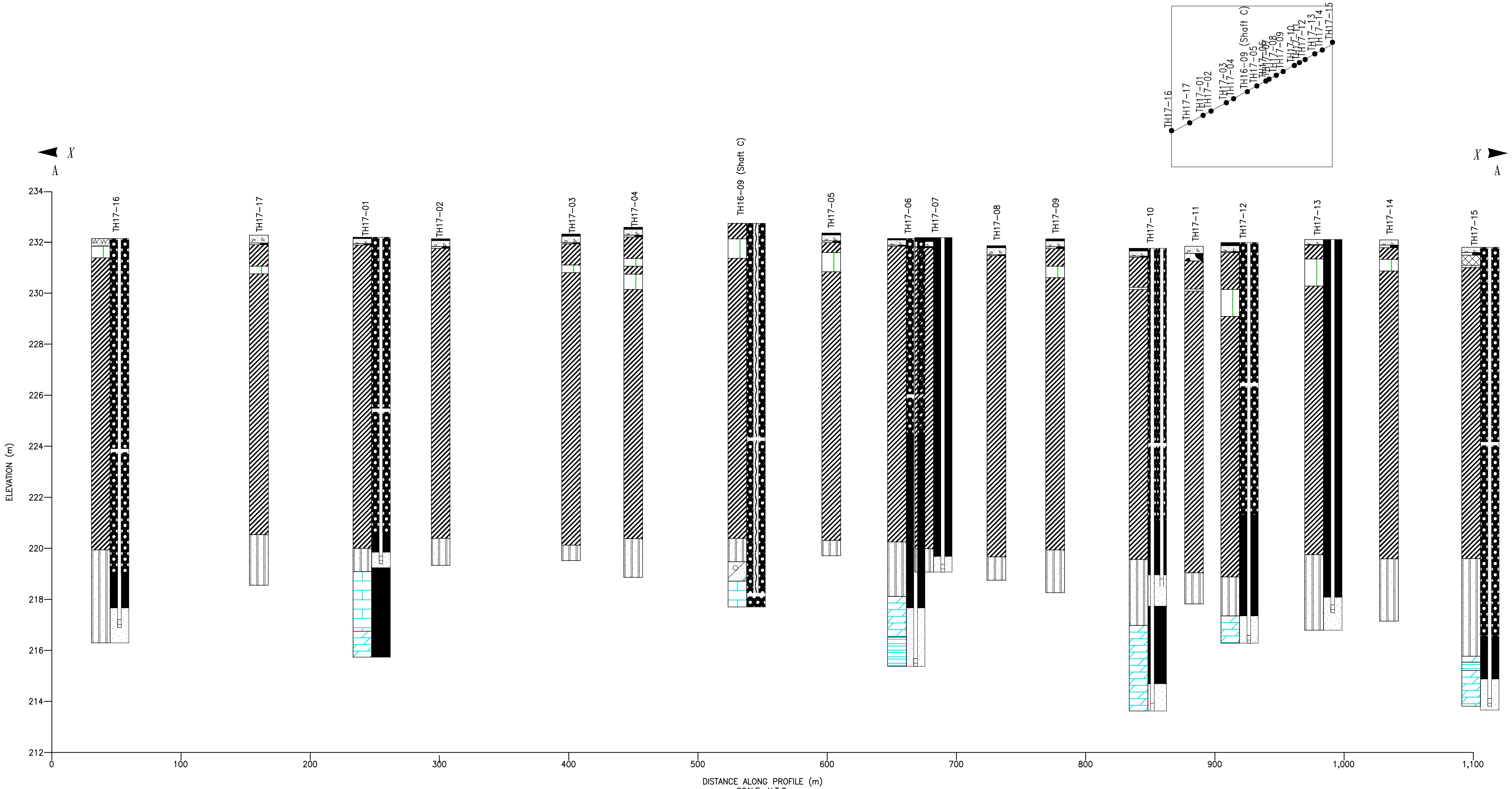
TABLE 3
GLACIAL TILL – SPT SUMMARY

Density	Frequency
Very Loose (0-4 blows/0.3 m)	
Loose (4-10 blows/0.3 m)	3
Compact (10-30 blows/0.3 m)	3
Dense (30-50 blows/0.3 m)	1
Very Dense (greater than 50 blows/0.3 m)	
Spoon Refusal (greater than 50 blows for less than 0.3 m)	13

TABLE 4
LIMESTONE BEDROCK – RQD SUMMARY

Rock Quality Designation	Frequency
Very poor (0-25%)	6
Poor (26-50%)	1
Fair (51-75%)	2
Good (76-90%)	3
Excellent (91-100%)	1

FIGURES



0 68

HORIZONTAL SCALE (METERS) VERTICAL EXAGGERATION: 23.5x

LITHOLOGY GRAPHICS

- INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
- INORGANIC SILTS AND SANDY SILTS OF SLIGHT PLASTICITY, ROCK FLOUR
- SILT TILL
- CLAY TILL WITH SAND MIXTURE/SANDY CLAY TILL
- LIMESTONE
- ASPHALT
- CONCRETE
- WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
- DOLOMITE
- SHALE
- FILL (MADE GROUND)
- WOOD FRAGMENTS
- TOPSOIL

LEGEND

TH17-01 BOREHOLE NUMBER

BOREHOLE LITHOLOGY

NO.	YY/MM/DD	DESCRIPTION	DESIGN BY	DESIGN CHECK
1	19/01/21	RE-ISSUED WITH FINAL REPORT	JRM	DAA
0	17/11/01	ISSUED WITH FINAL REPORT	JRM	DAA

REVISIONS / ISSUE

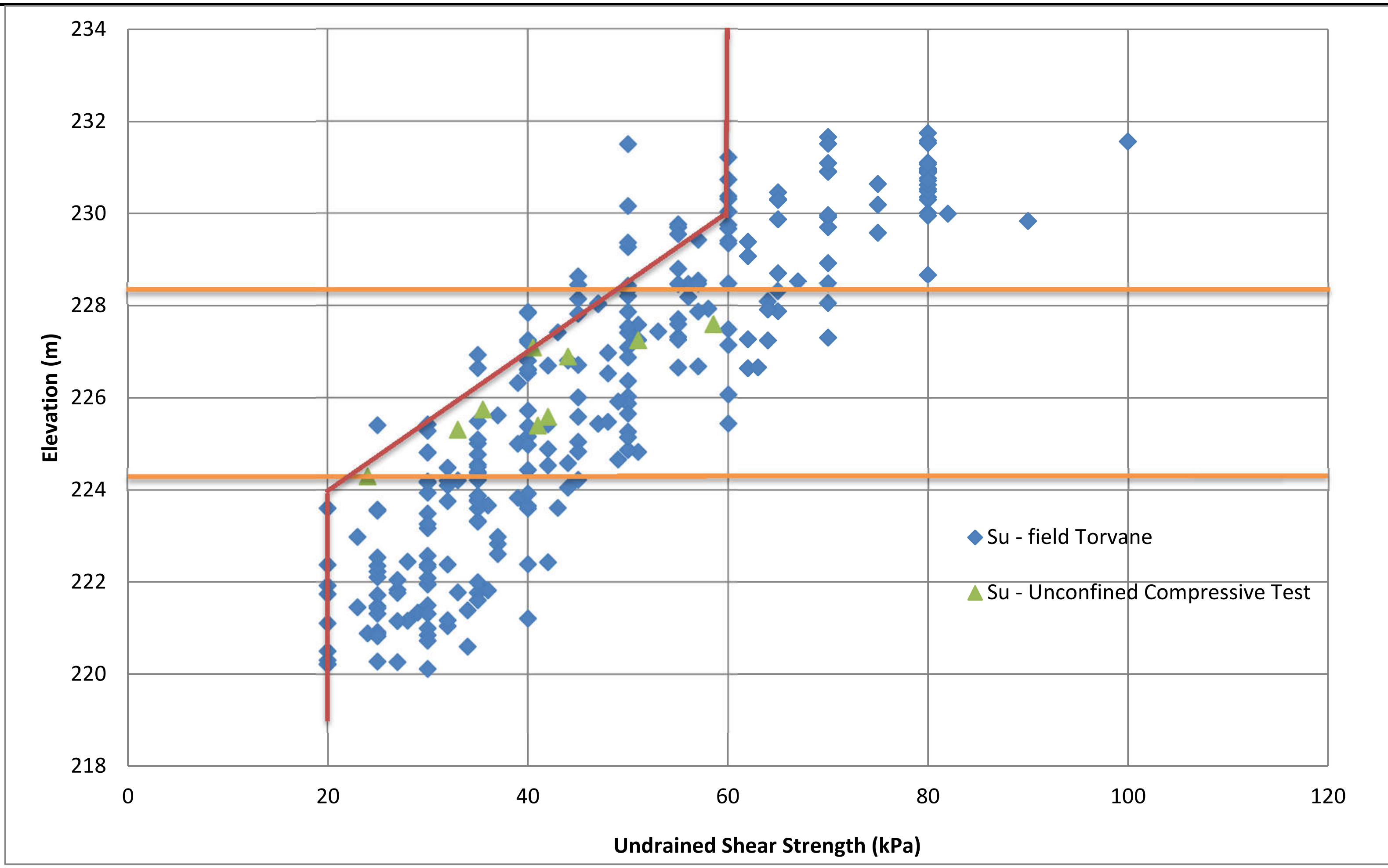
KGS GROUP CONSULTING ENGINEERS

Winnipeg

COCKBURN/CALROSSIE COMBINED SEWER RELIEF CONTRACT 5
WINNIPEG, MB

STARTIGRAPHIC PROFILE
2016 AND 2017INVESTIGATION

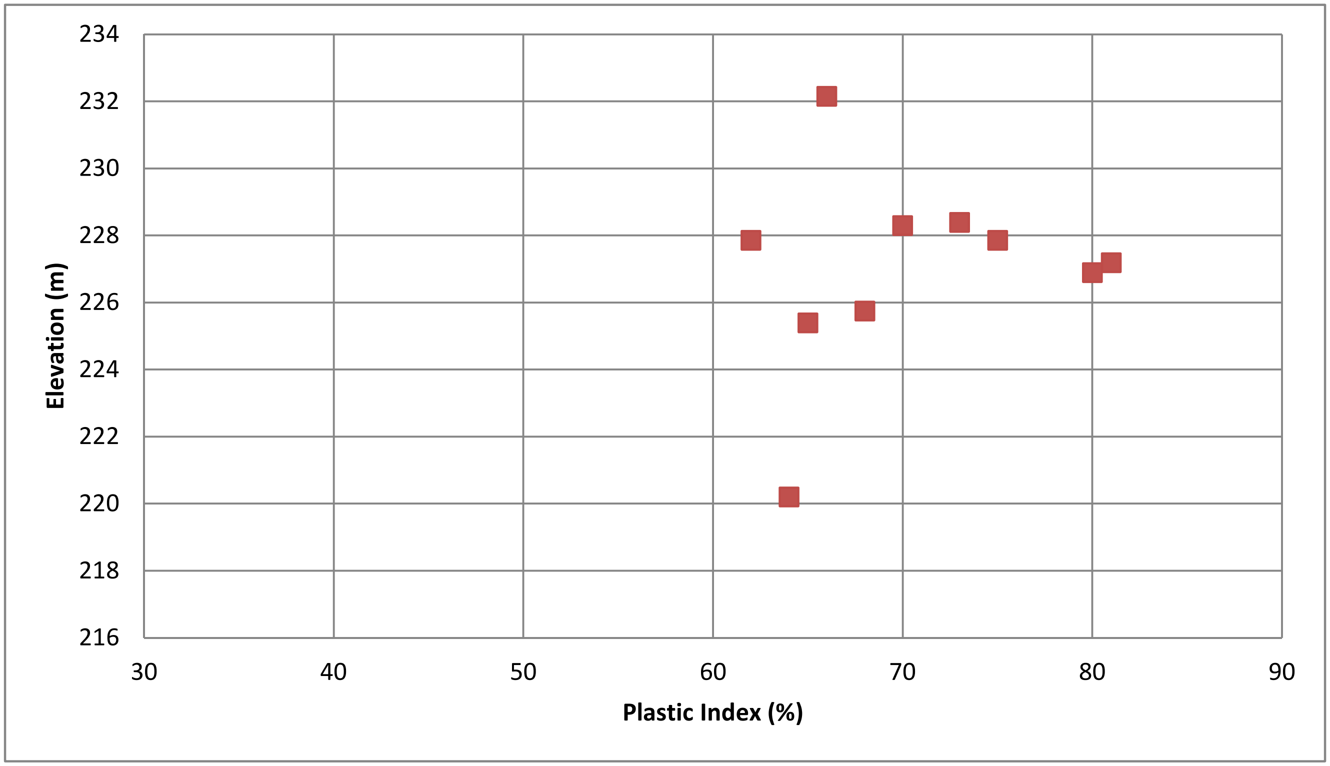
JANUARY 2019	FIGURE 01	REV: 1



NOTES:

- Baseline undrained shear strengths
- Approximate tunnel elevation

KGS GROUP	Winnipeg	
C5 - Cockburn/ Calrossie Combined Sewer Relief Project		
Geotechnical Baseline Report		
Undrained Shear Strength of Clay Deposit versus Elevation		
January 2019	Figure 2	1 Rev



NOTES:

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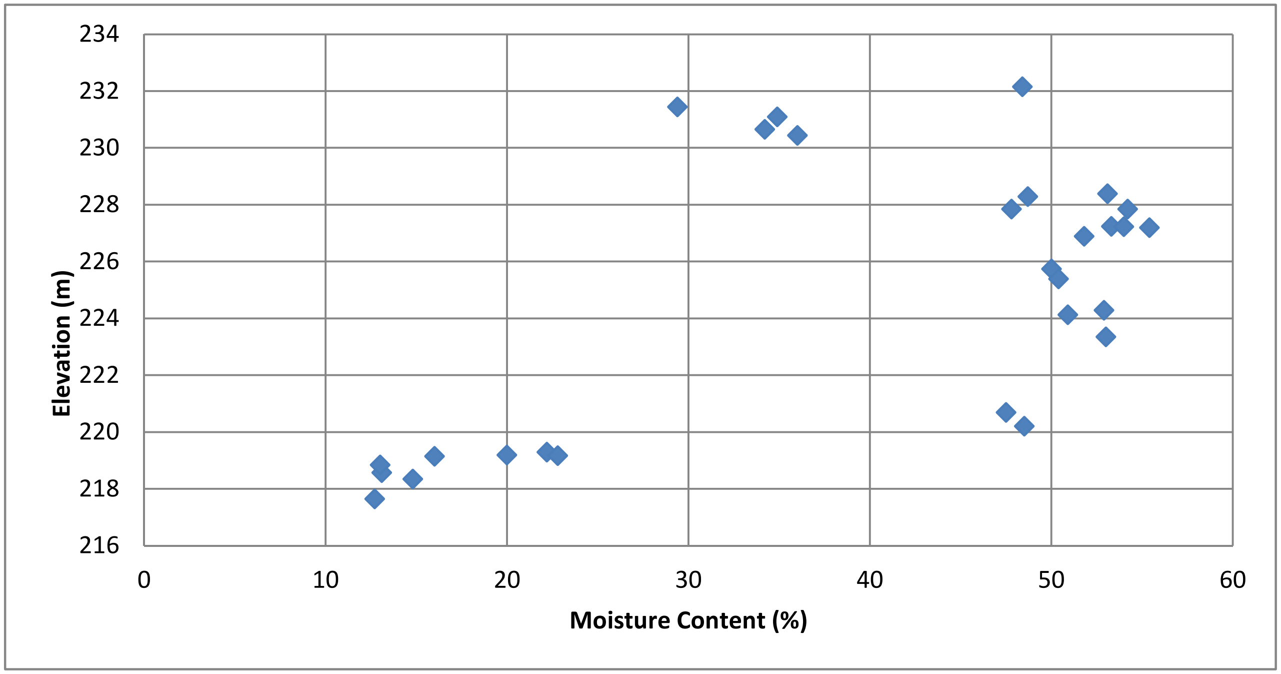
Geotechnical Baseline Report

Plasticity Index of Clay Deposit versus Elevation

January 2019	Figure 3	1 Rev



NOTES:

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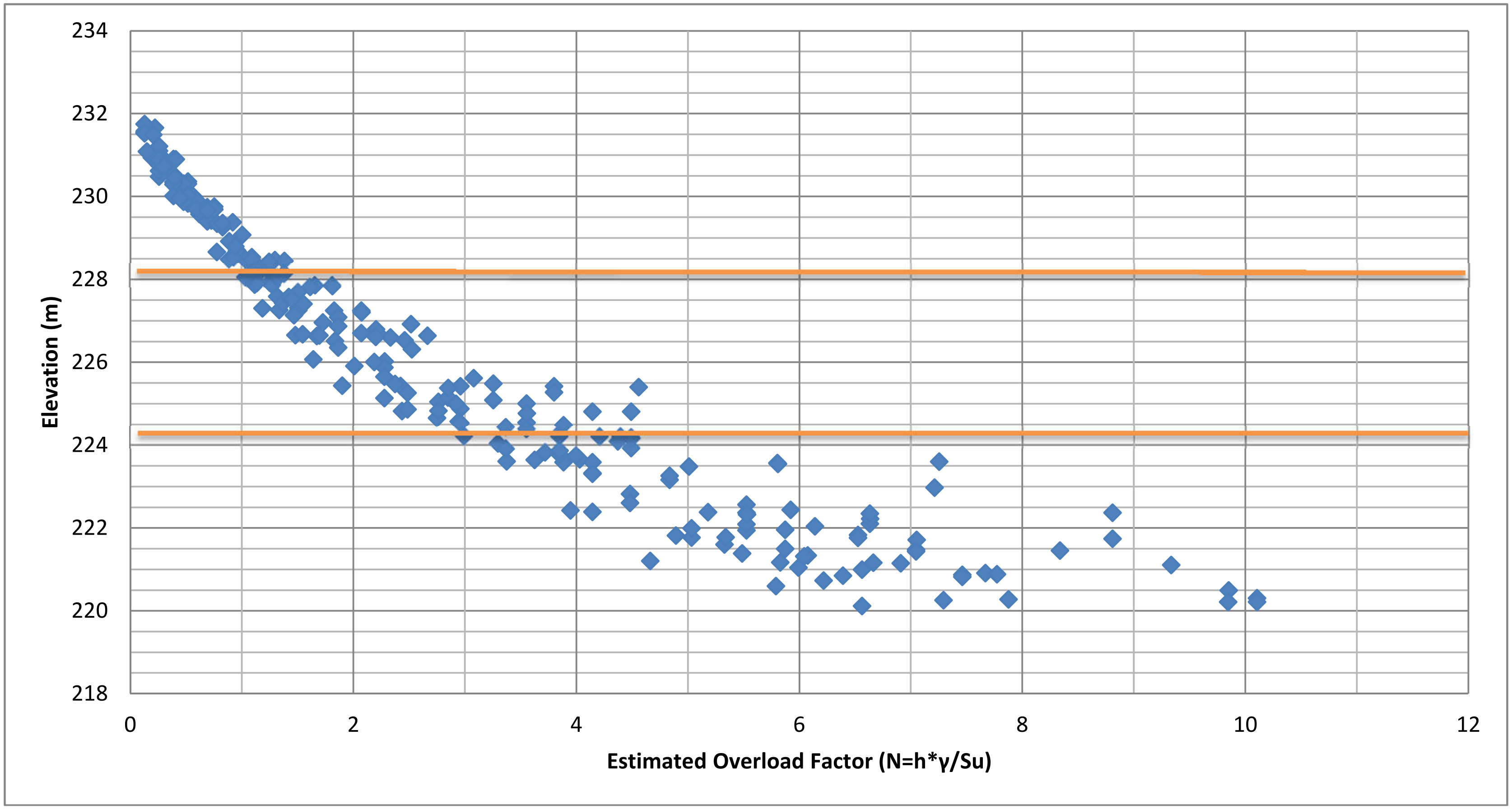
Geotechnical Baseline Report

Moisture Content versus Elevation

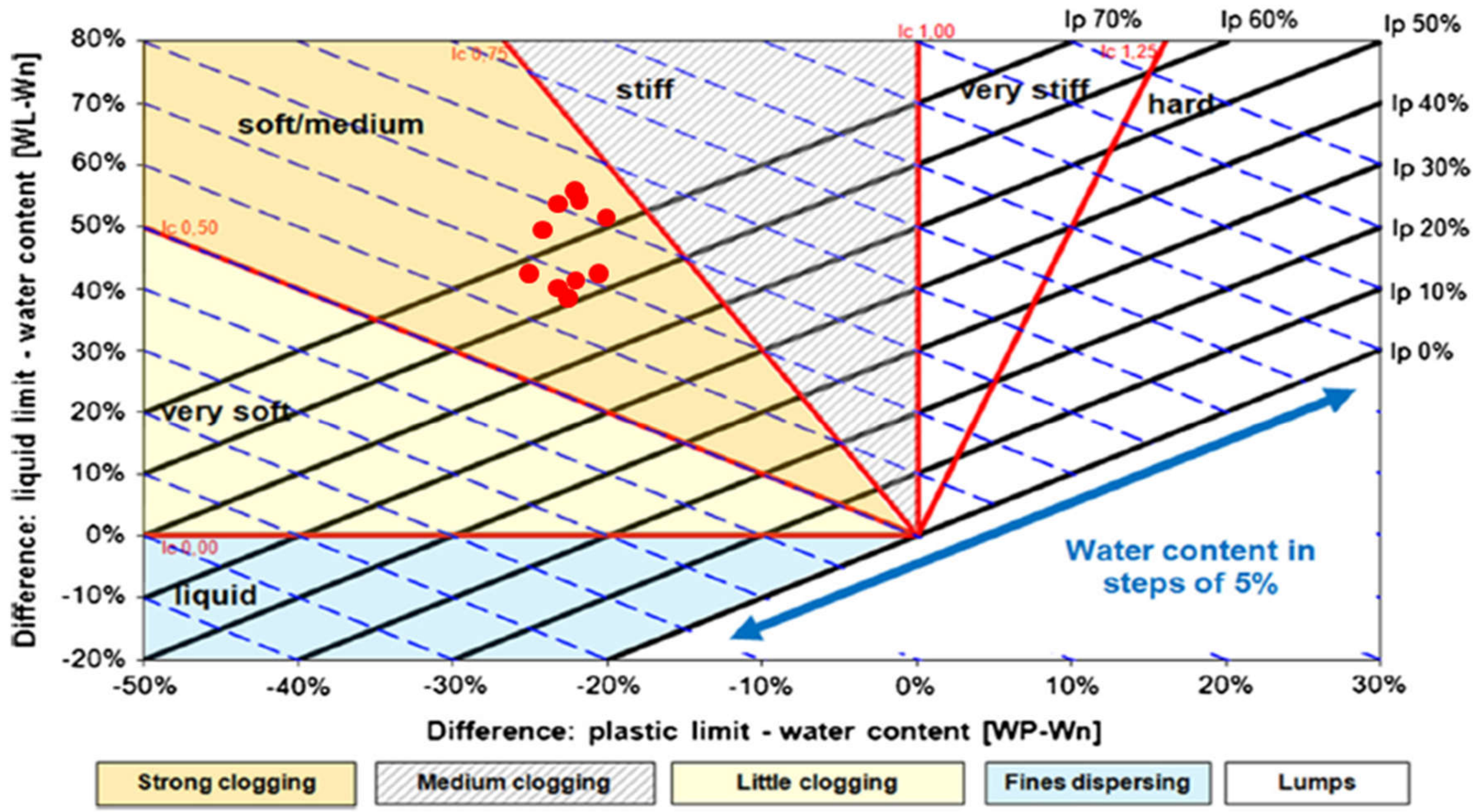
January 2019	Figure 4	1 Rev



NOTES:

Approximate tunnel elevation

KGS GROUP	Winnipeg	
Cockburn/ Calrossie Combined Sewer Relief Project		
Geotechnical Baseline Report		
Estimated Overload Factor (N) of Clay Deposit versus Elevation		
January 2019	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 Deposit		
January 2019	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)

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