CITY OF WINNIPEG

Newton Avenue Force Main Red River Crossing Geotechnical Baseline Report

Revision:
Final Rev 0

KGS Group Project:
22-0107-021

Date:
October 19, 2022

Client Project:
621-2021

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STATEMENT OF LIMITATIONS AND CONDITIONS

Limitations

This report has been prepared for City of Winnipeg in accordance with the agreement between KGS Group and City of Winnipeg (the "Agreement"). This report represents KGS Group's professional judgment and exercising due care consistent with the preparation of similar reports. The information, data, recommendations, and conclusions in this report are subject to the constraints and limitations in the Agreement and the qualifications in this report. This report must be read as a whole, and sections or parts should not be read out of context.

This report is based on information made available to KGS Group by City of Winnipeg. Unless stated otherwise, KGS Group has not verified the accuracy, completeness, or validity of such information, makes no representation regarding its accuracy and hereby disclaims any liability in connection therewith. KGS Group shall not be responsible for conditions/issues it was not authorized or able to investigate or which were beyond the scope of its work. The information and conclusions provided in this report apply only as they existed at the time of KGS Group's work.

Third Party Use of Report

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.

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 the site at the time of drilling. 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, KGS Group should be notified in order that the recommendations can be reviewed and modified if necessary.

1.0 INTRODUCTION

1.1 General

The City of Winnipeg Water and Waste Department is completing the replacement of one existing force main pipeline crossing the Red River between Fraser's Grove Park and Newton Avenue / Scotia Street in Winnipeg, Manitoba.

The new force main crossing will consist of 350 mm diameter pipe between Fraser's Grove Park and Kildonan Park; crossing beneath the Red River. The new force main pipe will be connected to the existing chamber in Fraser's Grove Park. Additional piping and chamber installation works are required to facilitate the force main connection from Kildonan Park to Newton Avenue. Horizontal Directional Drilling (HDD) construction methods will be employed for the installation of the proposed pipes.

1.2 Purpose of Report and Limitations

The primary purpose of this GBR is to set the anticipated geotechnical baseline conditions to be encountered during the construction of the proposed pipeline, as a common basis for bidding. This GBR presents an interpretation of geotechnical data collected during the project geotechnical exploration (KGS Group, 2021), including estimation /distribution of different materials to be encountered and the anticipated behaviour of these materials during pipeline construction. Baseline conditions described in this report provide a partial basis for the contractor to prepare construction bids and serve as the reference for the resolution of claims related to differing site conditions. Proponents must consider this GBR as part of the Contract Documents and it must be read in conjunction with the Specifications and the Design Drawings prepared by KGS Group for the City of Winnipeg. The hierarchy of this document and other documents is indicated in the Project's Contract Documents.

For the portion of the work affected by subsurface conditions, bids shall be based on baseline conditions presented in this GBR and the project plans/contract documents. Risks associated with conditions consistent with, or less adverse than the baseline conditions are allocated to the contractor. Those risks associated with conditions more adverse than the baseline conditions are accepted by the Owner. The provision of baseline conditions is not a warranty that baseline conditions will be encountered. These baseline conditions are rather the contractual standard that the Owner and the successful bidder will agree to use when interpreting differing or unusual site conditions. The owner accepts the risks for conditions that are less favorable than the stated baseline conditions and will negotiate with the contractor for additional compensation if these four conditions exist:

i. The contractor has demonstrated that they were able to perform the work within the baseline conditions prior to encountering a change in conditions.
ii. The actual conditions encountered are more adverse than baseline conditions.
iii. The contractor can document that the geotechnical conditions are more adverse than those described in this GBR and that exposed conditions materially and significantly increased the cost and/or time required to complete the work.

iv. The contractor has made diligent efforts to complete the work described in the contract documents, including any changes to methods, equipment, labor and materials made necessary by the more adverse conditions.

If all the foregoing conditions are met, then additional compensation will be negotiated as prescribed in the contract agreement. These general criteria shall be consistent with and negotiated in accordance with the contract's general terms and conditions. Notwithstanding the foregoing, nothing in this GBR shall invalidate or supersedes any of the terms and conditions of the contract agreement.

This Geotechnical Baseline Report (GBR) summarizes the geotechnical condition observed between Fraser's Grove Park and Kildonan Park along the proposed force main pipe alignment and provides construction considerations that form part of the basis of 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 and geophysical seismic refraction investigations carried out at the proposed site are presented in the Geotechnical Data Report (GDR) ("Newton Avenue Force Main Red River Crossing – Geotechnical Data Report – Draft – Rev A" KGS Group, 2022) which is included as Appendix A.

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 and bedrock have 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 and rock at the site.

The geotechnical data related to the subsurface conditions contained herein and in the GDR are intended for exclusive use of the City and the Contractor, if necessary, in evaluating the merits of differing site condition considerations 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 bidders confer with a qualified geotechnical engineer or engineering geologist who is familiar with all aspects of this report and the GDR. This engineer should have experience under 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.

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 Documents and Drawings for precise information on the dimensions and project layout.

2.2 Project Location

The project site is located in Winnipeg, Manitoba. The proposed force main pipeline runs from Fraser's Grove Park to Kildonan Park, and beneath the Red River 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 that 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 force main pipeline consists of a 350 mm internal diameter (ID) DR7 pipe from Fraser's Grove Park to Kildonan Park. The proposed pipeline alignment is approximately 460 m in length with an entry and exit angle of 18 degrees and includes a horizontal curve beneath the Red River. The proposed force main will tie into the existing chamber structure in Fraser Grove Park on the east side of the Red River. Additional underground linear infrastructure from Kildonan Park to Newton Avenue will be required to connect the new force main to the existing infrastructure. The proposed horizontal and vertical alignments of the force main pipeline including the length of curved and tangent sections, and invert elevations are shown on the Contract Drawings.

3.0 SOURCE OF INFORMATION

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

3.1 Geotechnical and Geophysical Investigations

1. KGS Group, April 2022. Newton Avenue Force Main Red River Crossing – Geotechnical Data Report – Draft Rev A.
2. Frontier Geoscience Inc., October 2021. Seismic Refraction Survey Report, Newton Force Main Red River Crossing, Winnipeg Manitoba.
3. AECOM Canada Ltd, March 2018. Northeast Interceptor Sewer – Red River Crossing Geotechnical Data Report.
4. AECOM Canada Ltd., April 2018. Northeast Interceptor Sewer Geotechnical Baseline 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. ASTM D 3035, Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Controlled Outside Diameter
3. ASTM F 512, Standard Specification for Smooth-Wall Poly (Vinyl Chloride) (PVC) Conduit and Fittings for Underground Installation
4. ASTM F 714, Specification for Polyethylene (PE) Plastic Pipe (SDR-PR) Based on Outside Diameter
5. ASTM F 1962, Standard Guide for Use of Maxi-Horizontal Directional Drilling for Placement of Polyethylene Pipe or Conduit Under Obstacles, Including River Crossings
6. ASTM F 2160, Standard Specifications for Solid Wall High Density Polyethylene (HDPE) Conduit Based on Controlled Outside Diameter (OD)
7. ASTM F 2620, Standard Practice for Head Fusion Joining of Polyethylene Pipe and Fittings
8. Canadian Associate of Petroleum Producers (CAPP), 2004. Planning Horizontal Directional Drilling for Pipeline Construction. CAPP Publication No. 2004-0022
9. Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition
10. International Society of Rock Mechanics, ISRM (1981). Suggested Methods for Rock Characterization, Testing and Monitoring. ISRM Commission on Testing Methods, Pergamon Press, Oxford.
11. City of Winnipeg, 2022. 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. Deere, D., 1964. Technical Description of Rock Cores for Engineering Purposes. Rock Mechanics and Engineering Geology, V.1, No. 1.
4. Department of Geological Engineering, University of Manitoba, 1983. Geological Engineering Report for Urban Development of Winnipeg.
5. Gamble, J.C., 1971. Durability-Plasticity Classification of Shales and Other Argillaceous Rocks. PhD Thesis, University of Illinois, Urbana.
6. Graham, J., and Shields, D.H., 1985. Influence of geology and geological processes on the geotechnical properties of plastic clay. Engineering Geology.

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 is outlined in the Geotechnical Data Report provided as Appendix A. 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 – 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. Canadian Geotechnical Journal 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. Skatfeld, 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, laboratory test data, and geophysical seismic refraction survey from the 2021 field investigations are presented in the GDR.

4.3 Geotechnical Investigations

A geotechnical investigation was performed in 2021 for the Newton Avenue Force Main Red River Crossing project. The investigation consisted of drilling at total of four (4) test holes including two (2) test holes located in Fraser's Grove Park, one (1) test hole within the Red River, and one (1) test hole in Kildonan Park.

Laboratory testing was performed on representative bedrock samples obtained from the geotechnical drilling investigation. Details of the 2021 field and laboratory programs are presented in the GDR including a compilation of geotechnical data obtained from the 2021 investigation and relevant projects located approximately 1.5 km of the site.

4.4 Groundwater Conditions

Groundwater level measurements obtained from the standpipe piezometers within the project area are summarized on Table 1. These measurements indicate that groundwater will be encountered during the pilot bore and reaming passes for the horizontal drilling and during excavation of the shafts for the installation of other linear underground infrastructure and connection to existing infrastructure.

TABLE 1: GROUNDWATER MONITORING DATA

Test Hole ID		TH21-01	TH21-03
Ground Elevation (m)		228.19	227.14
Piezometer Type		Standpipe	Standpipe
Tip Elevation (m)		211.40	205.74
Monitoring Zone		Glacial Till	Bedrock
Date	River Level		
9/10/2021	223.77(1)	222.20	222.58
10/28/2021	222.28(1)	223.29	223.11
5/5/2022	226.37(2)	224.19	225.01

Notes:
(1) River Level estimated at James Avenue approximately 7.5 km upstream of the project site.
(2) River Level estimated at Kildonan Bridge approximately 1.5 km downstream of the project site.

4.5 Geophysical Investigation

A geophysical seismic refraction survey was completed in 2021 along the proposed force main alignment. The objective of the geophysical survey was to obtain estimates of the depth to glacial till and bedrock along the preferred force main alignments. The results of the seismic refraction survey are summarized in a detailed seismic refraction report included in the GDR.

5.0 PREVIOUS HORIZONTAL DIRECTIONAL DRILLING CONSTRUCTION EXPERIENCE

Trenchless pipe installation using Horizontal Directional Drilling (HDD) methods in the City of Winnipeg has been prevalent for many years; however, there is limited local experience with installation involving river crossings. Challenges have been experienced in previous river crossing projects in the City of Winnipeg including the North Kildonan Feedermain Replacement project (City of Winnipeg Bid Opportunity 89-2015). A description of the work, challenges, and lessoned learned is outlined in the following Section.

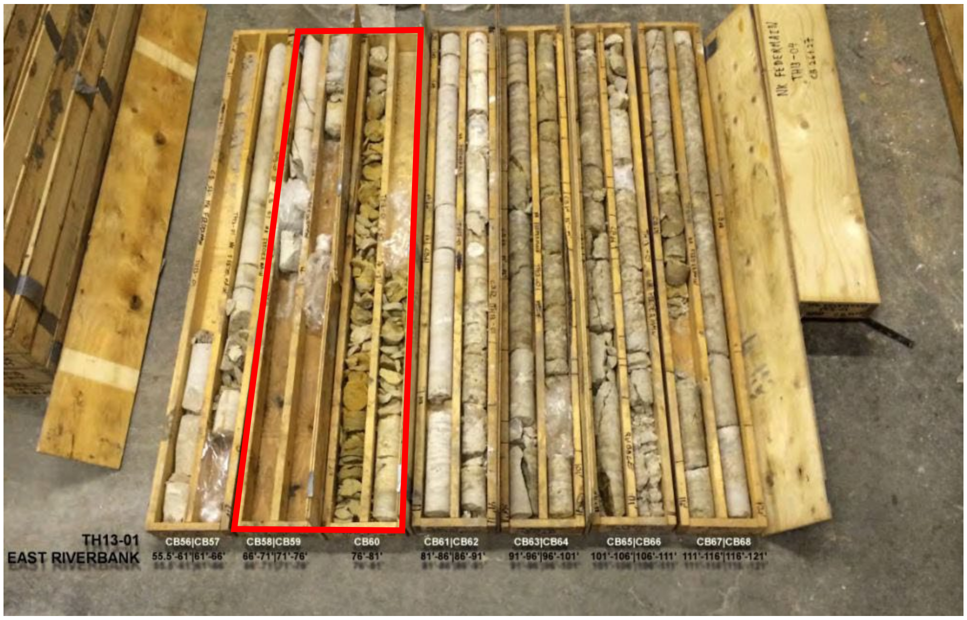
5.1 North Kildonan Feedermain Replacement

The North Kildonan Feedermain is a significant component of the City of Winnipeg's water distribution network. The feeder main is a 600 mm diameter steel pipe crossing the Red River that connects the Northeast and Northwest sections of the City of Winnipeg's water distribution network. In the fall of 2012, the City's Water Services Division determined the presence of a major leak in the feedermain, roughly at the midpoint of the 200 m long crossing. The following year, the City attempted to make a repair, however; the age and condition of the existing infrastructure made the process a challenge and the repair was not successful. At this point it was apparent that the feedermain would need to be rehabilitated or replaced to continue to provide reliable service for the City.

The replacement feedermain consisted of approximately 330 m of 750 mm diameter DR9 HDPE pipe installed under the Red River using HDD methods.

At the design stage, a geotechnical investigation was completed along the proposed feedermain alignment and supplemented by geotechnical data obtained from projects immediately upstream of the project site at the Kildonan Settlers Bridge. On the east side of the river, zones of highly fractured bedrock were encountered with long coring runs of little to no recovery from the drilling as shown in Figure 1, indicating a high probability of fluid / circulation loss to the formation in addition to the potential for hydraulic fracture.

FIGURE 1: NORTH KILDONAN FEEDERMAIN BEDROCK CORE SAMPLES



The in-channel borehole data from the Kildonan Settlers Bridge construction project was relied on for definition of the subsurface beneath the Red River; however, data limitations existed since the boreholes did not extend below the proposed invert for the feedermain. Initially, an in-channel borehole was planned specifically for the North Kildonan Feedermain project, but could not be completed due to water level constraints at the time of the drilling investigation. A geophysical seismic refraction survey was not performed either to develop a better understanding of the bedrock surface across the river. The Contractor was notified of the potential difficulties in the fractured bedrock formation.

Construction of the feedermain commenced in July 2015 and was completed in December 2015. The construction duration was atypical due to challenging ground conditions as well as Contractor operational challenges. The start of the HDD bore was behind schedule due to challenging casings installation within the overburden deposits. During the HDD bore, the Contractor experienced steering issues resulting in an approximate 1-2 week delay; lost a reamer in the drill hole resulting in an approximate 3-4 week delay; and pipe pullback required approximately 2-3 weeks.

Casing was installed through the overburden and seated into the bedrock at the entry side of the drill path, but not at the exit side. The most difficult formation to drill was the bedrock due to the highly fractured and weathered formation with poor recovery where the Contractor experienced difficulties with the pilot bore kicking off the design alignment. During reaming to the largest borehole diameter, one-side of the HDD tail string snapped, and the Contractor attempted to remove the reamer by pulling from the opposite side which also snapped. The reamer was unable to be retrieved, but there was no obstruction in the borehole when the Contractor attempted to ream the borehole again. It was unclear whether the broken reamer was lost into a large karstic void in the bedrock, or if a void was created from construction operations when attempting to retrieve the reamer following initial breaking of the tail string. Nevertheless, a new pilot bore was not required to successfully complete the feedermain installation.

In general, there were no major design changes from the tender to as-built stages. The Contractor primarily experienced challenges with steering the pilot bore potentially due to the fractured bedrock, but also within the overburden clay deposit. The as-built exit location extended an additional 12 to 15 m away from the design location.

The Contractor utilized a drilling mud weight of approximately 1200 kg/m^3 and did not experience high flows for cutting circulation (indicating potential loss of fluid to the bedrock formation as hydraulic fracture was not observed). Drilling fluid additives were primarily polymer and bentonite based. The Contractor had arranged for hauling of solid spoil to a farm or a City of Winnipeg landfill. The Contractor's reclaimer worked well to produce spoil in an acceptable state for transport to the farm location or to the landfill. Spoil that was too wet was disposed of using a hydrovac truck.

General lessons learned from the North Kildonan Feedermain project include the following:

- The quality of the limestone bedrock formation is highly variable, particularly in the weathered / altered zone. In-channel boreholes should be completed to below the proposed drill path alignment to improve understanding of the bedrock.
- Geophysical surveys can provide useful information related to undulations in the bedrock surface along the proposed drill path which can be used to optimize the installation of the overburden casings to ensure they can be seated well into the bedrock.
- Loss of circulation / drilling fluid to the bedrock formation due to the karstic nature of the local bedrock is probable. The drill path geometry was designed to mitigate the potential for hydraulic fracture to the watercourse. The Contractor experienced low return of drilling fluid, indicating potential losses to the bedrock formation.
- Challenges with steering were experienced in both the bedrock and the overburden (on the drill exit side). A shallow exit angle was difficult to control and resulted in an extension of the drill path and final pipeline length.
- The project scope of work required specialized and experienced contractors. The Request for Qualifications process was important to improve the confidence level for successful construction. The pre-qualified bids received for the project were initially above the City's budget and the project was split into multiple contracts with the river crossing portion being one contract.

6.0 SUBSURFACE CHARACTERIZATION

The general stratigraphy for the project site was developed based on the information obtained from the 2021 exploratory test holes supplemented with existing investigations from nearby sites, 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

In general, the stratigraphy consists of alluvium soils over lacustrine clay, glacial silt till and limestone bedrock. A thin layer of topsoil was encountered in test holes TH21-01, TH21-03, and TH21-04. An approximate 0.4 m thick layer of silty sand fill was encountered below the topsoil in TH21-01. Beneath the topsoil or fill material are alluvium and lacustrine soils, underlain by glacial silt till deposit and limestone bedrock. A simplified stratigraphic profile along the bore path is shown on the Contract Drawings.

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

- Topsoil;
- Fill;
- Alluvium soils;
- Lacustrine clay; and
- Glacial till

6.1.1 TOPSOIL

Topsoil was encountered at ground surface in the test holes drilled in Kildonan Park (TH21-01) and Fraser's Grove Park (TH21-03 and TH21-04), and was generally less than 300mm thick. The topsoil was black in colour and dry at the time of drilling

6.1.2 FILL

Silty sand fill was observed in test hole TH21-01 from elevation 228.1 to 227.7 m±. The silty sand fill was brown in colour, dry, loose in density, and contained medium to coarse grained sand.

6.1.3 ALLUVIUM SOILS

Alluvium soils ranging from sandy clay to sand were observed in test holes TH21-01, TH21-03 and TH21-04 at elevations ranging from 226.8 to 227.7 m± and extending to elevations ranging from 211.6 to 219.0 m±. Seepage is commonly observed within the alluvium soil layers.

Silty sand was observed in test hole TH21-03 from elevations 226.8 to 225.6 m± and in test hole TH21-04 from elevations 227.1 to 226.4 m±. The silty sand was brown in colour, dry, loose in density, and contained some silt.

Sandy clay was observed in test hole TH21-01, TH21-03 and TH21-04 from elevations 214.7 to 226.7 m±. The sandy clay was brown in color, damp, soft to stiff in consistency, of low to intermediate plasticity. The undrained shear strength within the sandy clay, as estimated using a field torvane on disturbed samples, ranged from 10 to 100 kPa and generally decreased with depth.

Clayey sand was encountered in test hole TH21-01, TH21-03 and TH21-04 from elevations 213.4 to 224.4 m±. The clayey sand was brown in colour, moist to wet, loose in density and contained fine grained sand. Interlayered sand and clay were observed throughout this layer.

Sandy silt was encountered in test hole TH21-04 from elevations 226.4 to 225.7 m±. The sandy silt was brown in colour, damp, of low plasticity, and contained some fine-grained sand lenses.

Sand was encountered in test hole TH21-01 from elevations 220.6 to 219.0 m± and in test hole TH21-03 from elevations 222.3 to 221.5 m±. The sand was brown to grey in colour, moist to wet, compact in density, and contained trace silt.

Alluvial clay (CI to CL) was encountered in test holes TH21-03 and TH21-04 from elevations 219.0 to 217.7 m±, and 216.5 to 214.9 m± respectively. The clay was grey in colour, moist, soft to firm in consistency, of low to intermediate plasticity, and contained trace sand. The undrained shear strength in the clay, as estimated using a field torvane on disturbed samples, ranged from 10 to 45 kPa.

Silt was observed at the base of the Red River in the test hole drilled in the river, TH21-02. The silt was grey, wet, very soft in consistency, and contained fine grained gravel. The silt was observed from elevations 217.7 to 216.6 m±.

A sand and gravel layer was encountered in test hole TH21-04 from elevation 213.4 to 211.5 m. The sand and gravel was grey in colour, moist to wet and dense.

The alluvium soils will be encountered during the pilot bore for the proposed force main pipeline and are susceptible to sloughing and caving. Casing is required through these soils, as shown on the Contract Drawings, to ensure the borehole remains open during drilling and installation of the product pipe.

6.1.4 LACUSTRINE CLAY

Lacustrine clay was encountered in test holes TH21-01 to TH21-03 overlying the glacial till at elevations ranging from 213.6 to 219.0 m±. The lacustrine clay ranged in thickness from 0.6 to 6.1 m. The clay was typically brown to grey in colour, damp to moist, firm to stiff in consistency and of high plasticity. In general, the consistency of the clay decreased with depth.

The material contained trace to some silt nodules. 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 of the clay deposit, as determined using a field Torvane on disturbed samples, ranged from 30 to 80 kPa and generally decreased with depth.

Fine to coarse grained gravel and boulders were encountered in the grey clay near the till interface.

The lacustrine clay will be encountered during the pilot bore for the proposed force main pipeline. Casing is required through these soils, as shown on the Contract Drawings, to ensure the borehole remains open during drilling and installation of the product pipe.

6.1.5 GLACIAL TILL

Glacial silt till was encountered below the clay and sand with gravel at elevations ranging from 211.6 to 212.9 m± in the test holes. The glacial till ranged in thickness from 3.1 to 5.8 m. The silt till was brown in colour, damp to moist, compact to very dense and contained some fine to coarse grained gravel and some fine to coarse grained sand. Boulders and cobbles are commonly found within the till layer and should be anticipated within the deposit at this project site.

The uncorrected Standard Penetration Test (SPT) blow counts for 300 mm ranged from 17 to greater than 50 blows, classifying the material as compact to very dense. The till was classified as very dense (greater than 50 blow for 300 mm) for one (1) of the SPTs. A summary of the uncorrected SPT N values recorded in the silt till are presented in Table 2 of this report.

TABLE 2: 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)	4
Very Dense (greater than 50 blows/0.3 m)	1
Spoon Refusal (greater than 50 blows for less than 0.3 m)	1

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 have been observed within the lacustrine clay layer during previous trenchless construction projects within Winnipeg. The new force main pipe will be installed primarily in the underlying bedrock and there is a low potential for being impacted by the cobbles and boulders within the glacial till deposit; however, the contractor should be aware that cobbles and boulders may be encountered during the initial pilot bore.

6.2 Bedrock Characterization

The limestone bedrock within the project site is the Selkirk member of the Red River Formation. The Selkirk member typically is medium strength with compressive strengths that vary from 30 to 40 MPa. The Young's modulus (E) generally ranges from 15 to 25 GPa (University of Manitoba, 1983). The bulk modulus (k) typically ranges from 40 to 50 GPa, and the shear modulus ranges from 5 to 10 GPa. The majority of the force main is intended to be installed within the bedrock.

Bedrock was encountered below the silt till at elevations ranging from 207.1 to 209.7 m± based on test hole drilling. The seismic refraction survey suggest that top of bedrock may be lower on the east side of the river, at an elevation of approximately El. 198 m± along the proposed force main alignment. The estimated bulk compressive wave velocity (Vp) for the upper bedrock is 4100 m/s and 3200 m/s on the east side and west side, respectively. These estimated velocities suggest that the bedrock is more fractured on the west side.

The bedrock consists of limestone and mottled limestone. Dolomite was observed in test hole TH21-01 from elevations 208.0 to 209.7 m±. The measured RQD and total core recovery (TCR) of the bedrock with elevation is summarized in the GDR.

The dolomite was brown in colour, and fine grained. Weaker fractured rock with closely spaced joints was generally observed above elevation 208 m±. Shale was observed at elevation 208.0 m±. The rock quality designation (RQD) of the dolomite was 62, classifying the rock as fair.

Limestone was generally encountered below elevations of 208.0 m±. The limestone was white to grey colour, and medium grained. A soft clay seam 50 mm thick was observed in test hole TH21-01 at elevation 207.0 m±. The RQD of the limestone ranged from 21 to 91. In general, the RQD was greater than 80 below elevation 205 m±, classifying the rock as good to excellent.

Mottled limestone was encountered in all test holes at elevations ranging from 203.7 to 207.9 m± and extending to the end of the test holes. The mottled limestone was mottled white, brown and grey in colour, medium grained and strong. The jointing was moderate to wide spaced. Weak zones of soft clay seams up to 50 mm were noted within the mottled limestone in test hole TH21-01 between elevations 203.3 and 197 m±. The RQD of the mottled limestone ranged from 75 to 100, generally increasing with depth. In general, the RQD was greater than 90 below elevation 197 m±, classifying the bedrock as excellent.

Laboratory testing was completed on two (2) mottled limestone bedrock samples from test hole TH21-01, at elevations 200.5 and 202.7 m±. The compressive strength was measured to be 14.4 and 28.4 MPa in the upper and lower samples respectively, classifying the rock as weak to medium-strong. The Young's Modulus was measured to be 12.1 and 19.3 GPa and the Shear Modulus was calculated to be 5.4 and 8.3 GPa in the upper and lower samples respectively.

Karst openings are commonly encountered in limestone and dolomite formations around Winnipeg; these feature are results of bedrock solution processes and can also be a source of loss of circulation and mud control problems during horizontal directional drilling. Karst voids may be encountered within the limestone bedrock along the proposed pipeline alignment even though no extensive karst features were observed in the boreholes that were drilled at the site. However, the overall risk of encountering these features is low to moderate based on the RQDs and the bedrock quality obtained from the 2021 investigation program.

The limestone bedrock joints/fractures can also result in migration of drilling fluid (loss of circulation) and instability of the borehole. The possible occurrence of cobbles and boulders within glacial till soils above the bedrock is another fissure that could provide paths for fluid to migrate out of the bore path. However, these risks may be mitigated by using drilling additives to consolidate and reduce the permeability of joints and fractures.

6.3 Groundwater Conditions

The relationship between the Red River and the underlying Carbonate Aquifer is very dynamic, and typically the potentiometric surface of the aquifer is above the river level. Aquifer levels in the summer are characterized by a reduction in groundwater levels in the Carbonate Aquifer which tend to fall below the river level. The aquifer begins to rise during the fall as river levels decrease (Render, 1970) (AECOM, 2018).

Flood elevations for the Red River at the project site are shown in Table 3.

TABLE 3: SUMMARY OF RED RIVER FLOOD EVENT ELEVATIONS

Return Period	River Flood Elevation (m)
1:2 Year	225.44
1:5 Year	226.16
1:10 Year	226.24
1:50 Year	227.20
1:100 Year	228.14

A total of two (2) standpipes piezometers were installed at the project site. One (1) standpipe was installed in the glacial till at TH21-01; and one (1) standpipe was installed in the bedrock at TH21-03. The measured groundwater levels are presented in the Geotechnical Data Report for the project (Appendix A).

6.3.1 BASELINE GROUNDWATER LEVELS

Groundwater level fluctuate with the river level and 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 4. The baseline groundwater level for the overburden soils correspond to the Red River level for a 1 in 100-year flood to account for potential seasonal fluctuations and known interconnection of the till deposits and bedrock with the Red River. The bedrock baseline ground elevation was increased by an additional 0.5 m.

TABLE 4: BASELINE GROUNDWATER LEVELS

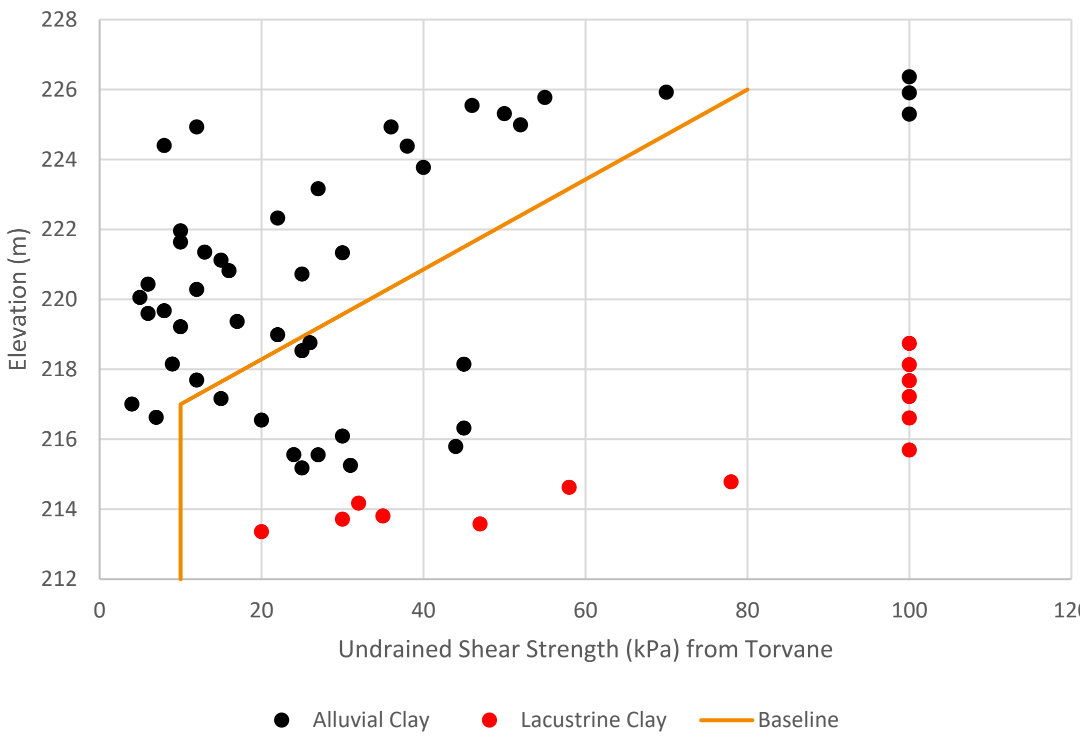
Soil Strata	Groundwater Elevation (m)
Overburden Soil (including alluvial deposits and glacial till)	228.1
Bedrock	228.6

6.4 Baseline Values

6.4.1 OVERBURDEN

- For baseline purposes, the undrained shear strength of the cohesive clay soils varies from 80 kPa at El. 226 m to 10 kPa at 217 m. The undrained shear strength of the clay is 10 kPa below El. 217 m and 100 kPa above El. 226 m. These strengths are representative of clay at its natural moisture content. The measured shear strengths and baseline undrained shear strengths are shown on Figure 2.

FIGURE 2: UNDRAINED SHEAR STRENGTH WITH ELEVATION FOR COHESIVE SOILS



- The baseline effective shear strength parameters and permeability of each soil strata are outlined in Table 5.

- For baseline purposes, the range of Liquid limits of the clay is 25% and 75% and the range of Plastic limits of the clay is 15% to 20%. The corresponding baseline plasticity index varies between 15% and 50 %.

TABLE 5: BASELINE EFFECTIVE SHEAR STRENGTH PARAMETERS

Material Type	c' (kPa)	Φ' (degrees)	ɣ (kN/m^3)	K_{sat} (m/sec)
Silts	5.0	14	18.5	$1x10^{-7}$
Upper Brown Clay / Alluvial Clay	5.0	14	18.5	$1x10^{-9}$
Lower Grey Clay	5.0	14	18	$1x10^{-10}$
Glacial Till	5.0	23	22	$1x10^{-7}$

6.4.1.1 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 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.

6.4.2 BEDROCK

- For baseline purposes, the RQD of the bedrock varies from 20% to 70% at the top of bedrock between El. 210 m and El. 203 m. RQD below El. 203 ranged between 90% and 100%.
- For baseline purposes, the hardness of the bedrock in accordance with ISRM (1981) is interpreted to be weak to medium strong based on unconfined compressive strength tests with values ranging from 14 to 28 MPa (approximately 2000 to 4000 psi).
- For baseline purposes, the Young's Modulus (E) of the bedrock ranges from 10 to 25 GPa; Poisson's ratio ranges from 0.1 to 0.2; and the Shear Modulus ranges from 5 to 10 GPa based on testing completed on bedrock samples from the site.

7.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

7.1 Trenchless Pipe Installation Methods

The Contractor is to install the new force main pipe using Horizontal Directional Drilling (HDD) methods subject to the detailed requirements of the technical specifications.

The HDD machine and tooling 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 limitations at the site and other constraints that have been identified in the Contract Documents.

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

7.2 Mixed Ground Conditions

The Contractor can expect to encounter mixed ground conditions along the drill path including alluvium soils, glaciolacustrine clay, glacial till deposit, and weathered to competent bedrock as identified in the GDR. The Contractor shall ensure that the HDD equipment and tooling selected is capable of navigating these mixed ground conditions.

7.3 Entry and Exit Pits

A minimum of one (1) entry pit and one (1) exit pit is anticipated to be excavated for the installation of the force main pipe as shown on the Construction Drawings. Workspace needed for an HDD bore of this scope is anticipated to be 40 m by 60 m for the entry area, 20 m by 20 m for the exit area, and a 20 m wide area the length of the drill section for the pipe preparation and fusing area.

Fraser's Grove Park is well suited for the entry area. The area long Rainbow Drive and McKay Drive within Kildonan Park space provides a large open space in the park to serve as an exit area. The open green space within Kildonan Park provides suitable workspace to stage the project pipe.

Open cut connections to the existing underground infrastructure are required on either end of the HDD crossing. On the east side of the Red River, a short open cut connection is required to tie-in to the existing chamber in Fraser's Grove Park. On the west side of the Red River, a new section of force main will be installed along Scotia Street to connect to the sanitary trunk at Newton Avenue.

7.3.1 CONDUCTOR CASINGS

Conductor casings are required for this project to maintain the stability of the borehole within the overburden deposits. When drilling through bedrock, high fluid flows are typically required to carry the cutting created the drilling and reaming process. This can erode the softer overburden material resulting in a collapsed borehole and loss of fluid circulation. The conductor casings shall be steel pipes sized to accept the largest reamer expected (as specified in the Contract Documents) and are embedded an appropriate depth into the bedrock to ensure a tight seal and enable the slurry to return to the rig. Casings will be straight tangent sections which extend from the surface to a short distance inside the bedrock as indicated in the Contract Documents.

7.4 Horizontal Directional Drilling

General design and construction considerations for the HDD bore are outlined below:

- The Contractor is responsible to select the appropriate equipment and tooling based on the geotechnical conditions outlined.
- The HDD bore will be drilled through the overburden and into the bedrock formation to approximately 31 m below the river thalweg.
- The properties of the overburden and bedrock are outlined in Section 6.0 and the GDR.
- The Contractor is required to collect and discharge groundwater flows according to the Contract Documents.

7.4.1 HYDRAULIC FRACTURE PLAN

The Contractor will be required to develop a hydraulic fracture plan to mitigate the release of drilling fluid to the ground surface and Red River. The minimum requirements for the hydraulic fracture plan are outlined in the Contract Documents.

7.5 Temporary Excavations

Temporary excavations will be required to facilitate the construction of the proposed force main pipeline. All excavation work is 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. 217/2020.

Baseline groundwater levels are presented in Table 4. Baseline soil strengths for temporary excavation design are outlined in Table 5.

Excavations performed adjacent to the existing roadway or other existing 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 registered with Engineers Geoscientists Manitoba (EGM) with appropriate qualifications. 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.

Some overburden deposits common to Winnipeg 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.

7.6 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 systems 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 documents. The Contractor will maintain specified clearances from buried utilities and infrastructure as indicated in the Contract 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 experience 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.

7.7 Instrumentation Program

The Contractor is required to monitor the HDD bore as indicated in the Contract Documents

7.8 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 follow the 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.

7.9 Frost Penetration

The expected depth of frost penetration has been estimated assuming a design freezing index of 2680°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.

7.10 Corrosion Potential

The degree of exposure of concrete in contact with soils to sulphate attack is classified in CAN/CSA A23.1-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.

APPENDIX A

Newton Avenue Force Main Red River Crossing

Geotechnical Data Report

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