AECOM

Replacement of the FGSV Siphon

Geotechnical Baseline Report
FINAL

City of Winnipeg

607228226

May 2025

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AECOM: 2015-04-13

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1. Introduction

1.1 General

AECOM Canada ULC was retained by the City of Winnipeg (the City) to develop a geotechnical baseline report (GBR) for the proposed replacement of the Fort Garry - St Vital (FGSV) Siphon crossing of the Red River. The FGSV Siphon receives wastewater flows for the D'Arcy Lift Station servicing approximately 3,360 ha of development in the southwest section of the City. The project site is located in south end of Winnipeg, MB, on Abinojii Mikanah adjacent to the Fort Garry Bridge.

1.2 Purpose of Report and Limitations

AECOM has prepared this Geotechnical Baseline Report (GBR) for the Replacement of the FGSV Siphon across the Red River, located at the Fort Garry Bridge in south Winnipeg, Manitoba.

This GBR is intended to apply to the proposed river crossing only, located south of the east bound Ft Garry bridge, including the two tunnel shafts and the pipe located between the two shafts. Other aspects of this project including gravity sewers extending from and to the proposed siphon location, connections to existing pipes and structures, and modifications of the existing overflow structure are not subject to the baselines included in this report.

The purpose of this GBR is to:

- Provide a baseline interpretation of the geotechnical of the works;
- Set clear baselines for subsurface conditions anticipated to be encountered during construction;
- Provide all bidders with a single contractual interpretation in preparing bids;
- Describe the subsurface conditions along the FGSV Siphon alignment; and,
- Assist in evaluating the requirements for excavation, temporary support, groundwater control, and ground movement for shaft and tunnel construction.

The GBR presents the subsurface conditions as baseline values and descriptions that the contractors shall use for their tenders. The GBR should be read in conjunction with the Geotechnical Data Report (GDR) prepared for FGSV Siphon by AECOM dated April 2025. The baselines presented in this GBR do not provide a warranty that subsurface conditions different from the baselines will not be encountered. The baselines, however, represent a contractual agreement between the City of Winnipeg (the City) and the Contractor to use for the resolution of claims made for "differing ground conditions". Contractors 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 AECOM for the City. The hierarchy of this document and other documents is indicated in the Project's Contract Documents. The baselines presented in this GBR apply to the excavation limits shown on the Design Drawings and Figures provided in this GBR. The baselines presented in this GBR do not apply to Contractor-modified portion(s) of the Project.

The baselines in this GBR also provide the City with the opportunity to allocate risks associated with the variability in the subsurface ground conditions during the bidding stage. Risks associated with consistent or less adverse subsurface conditions than baselined subsurface conditions are allocated to the Contractor and risks associated with more adverse subsurface conditions than the baselined subsurface conditions are accepted by the City. The effective use of the baseline conditions will depend on adequate documentation of subsurface conditions encountered during trenchless utility installation.

This GBR has been prepared in general conformance with the guidelines and practices described in the Geotechnical Baseline Reports for Construction, Suggested Guidelines, published by ASCE, 2022. The GBR has been prepared by AECOM for the City. Some of the technical concepts, terms and descriptions in this GBR may not be fully understood by bidders. It is required that bidders have a geotechnical engineer with local experience, who is familiar with the topics in this GBR, to carefully review and explain this information so that a complete understanding of the information presented in this GBR can be developed prior to submitting a bid.

Certain elements of the Project are based on requirements that cannot be varied unless otherwise specified in this GBR. These include, but are not limited to, the following:

- Use of full supported face during tunneling
- Mixed face conditions are expected.
- Adoption of ‘sealed’ methods of shaft construction – ‘sealed’ methods of shaft construction may include secant piles, pre-cast concrete or cast-in-place concrete caissons, or other methods. All sealed shafts are required to have a concrete base designed to prevent basal heave, resist hydrostatic pressures, and minimize ingress of fines and infiltration of groundwater.
- Microtunnelling Boring Machine (MTBM) launch and receiving shafts minimum dimensions to support proposed control structures.
- Final minimum siphon internal diameter.
- Alignment of pipes and incoming and outgoing trunk inverts for the proposed siphon.

Other elements of the project that are flexible and afford the Contractor latitude in planning its work and selecting means and methods, include, but are not limited to, the following:

- Procurement, selection, and configuration of the Microtunnel Boring Machine (MTBM).
- Design of the jacking pipe, although there are minimum requirements that must be satisfied.
- Type of sealed shaft support system.

2. Project Description

2.1 General

The descriptions and dimensions for the various components of the project provided in this GBR are approximate and for illustration purposes only. The Contractor should refer to the Contract Documentations/Drawings for accurate information on dimensions and project layout.

2.2 Project location

The project site is in the southern part of Winnipeg near the existing Fort Garry Bridge on Abinojii Mikanah. The proposed FGSV Siphon alignment will cross the Red River directly south of the east bound bridge.

2.2.1 Adjacent Structures

A high-rise residential development is located approximately 20 to 50 meters southeast of the siphon outlet chamber located east of the Red River. Additionally, a residential neighborhood is situated approximately 70 to 80 m southwest of siphon inlet chamber located west of the Red River. The Fort Garry Bridge is located approximately 30 m north of the proposed FGSV alignment. These structures are not directly above the siphon alignment, and therefore, settlement is not a concern for these structures.

Multi-use paths are located on both the eastern and western embankments. These paths are directly above the FGSV Siphon alignment, making settlement a potential concern for these structures.

The existing siphon sewer alignment is located directly north of the proposed FGSV Siphon alignment, in between the two bridges. Overhead electrical utility lines run near parallel to the existing siphon alignment at the Red River Crossing. Additional existing buried utilities, such as Telus fibre lines, are present just south of the Fort Garry Bridge, which is parallel and aligns with the proposed FGSV Siphon alignment and crosses the Red River. However, these structures are not directly on top of the siphon alignment. Settlement is not a concern to these structures.

The proposed FGSV alignment relative to adjacent and pertinent features is shown in **General Plan** within the **Contract Documents and Drawings**.

2.2.2 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.

The Red River levels vary significantly throughout the year, with notable differences in ranges:

- Spring: Highly variable up to 230.89 mASL (1:700 year flood).
- Summer: approximately 223.98 mASL.
- Winter: approximately 221.76 mASL.

For more details regarding the river levels, see Section 6.4.1.

2.3 Key Components of the Project

The FGSV Siphon replacement project aims to replace the failed 700 mm and 800 mm wastewater siphons that cross the Red River between the Fort Garry eastbound and westbound bridges. It is expected that construction will start with the construction of a launch shaft at the siphon outlet of a launch shaft at the siphon outlet chamber, where the micro tunnel will exit at the siphon inlet chamber.

The new FGSV siphon replacement will be installed using a trenchless method, specifically utilizing micro tunnel boring machine (MTBM) technology. This method involves tunneling underneath the river, starting at the launch shaft located at an elevation of approximately **216.40 m** (near testhole TH24-05) and exiting at the receiving shaft at an elevation of approximately **222.7 m** (near testhole TH24-01). This approach allows for minimal disruption to surface activities and infrastructure while efficiently replacing critical underground infrastructure.

- **Shaft Details**: The launch and receiving shafts will have a minimum diameter of approximately **10.0 m** to suit the final siphon chamber configuration. These dimensions may be adjusted based on the Contractor’s equipment, construction methodology and the lengths of the jacking pipes selected.
- **MTBM Technology:** A large **2100 mm diameter reinforced concrete pipe (RCP)** casing will be installed beneath the river in bedrock using MTBM. Two **900 mm DR11 high-density polyethylene (HDPE)** carrier pipes will be pulled through after the casing installation. The invert elevation of the RCP is expected to be approximately **206.31 m**, with a bore path consisting of a Launch and exit angle of 9 **degrees** and a **500 m bending radius**, covering a shaft-to-shaft distance of approximately **350 m**.

The scope of work of this Project includes:

- Site mobilization and establishment of work areas.
- Installation of MTBM launch and receiving shafts.
- Installation of approximately 350 long river crossing (siphon) using Microtunneling:
    - 2100 mm internal diameter primary casing pipe through underlying limestone bedrock strata.
    - Two (2) 900 mm DR 11 HDPE carrier pipe to be pulled through casing pipe on casing spacers.
- Conversion of the launch and receiving shafts into final control chamber configuration:
    - Installation of chamber foundation and walls (if not part of construction shafts).
    - Installation of permanent roof and service access projection to grade.
    - Installation of intermediate floor(s), ladders, lighting, and other man-entry accommodations.
    - Installation of chamber appurtenances.
- Site restoration works.

Details of the alignment and elevations are illustrated in the **General Plan** within the **Contract Documents and Drawings**.

3. Local Trenchless Construction Experience

3.1 General

Select case histories relevant to the current project's design and construction, and lessons learned from microtunneling construction using MTBM in the Winnipeg area are presented in the following sections.

3.2 Northeast Interceptor Sewer Project

The Northeast Interceptor Sewer (NEIS) project, located in the Kildonan area of northeast Winnipeg, involves the construction of a new sewer alignment to address capacity issues and surcharging during severe wet weather events. The proposed alignment crosses the Red River just south of the Kildonan Settlers Bridge and runs almost parallel to the existing siphon sewer. Key components of the project include the installation of a 1200 mm carrier pipe using microtunneling methods and the construction of inlet and outlet chambers on both riverbanks. Additionally, the project utilized vertical curves to minimize shaft depth and rock excavation within shafts, sealed shaft, 1500 OD RCP, and sunk concrete caisson shaft construction that is found on top of bedrock. The project also involves navigating various adjacent structures and utilities, such as a high-rise residential development, the Kildonan Golf Course, and existing utility lines.

General Lessons learned from the Northeast interceptor Sewer Project include the following:

- **Karstic Features:** The Geotechnical Data Report (GDR) and Geotechnical Baseline Report (GBR) should characterize if karstic features are or could be present.
- **Fractured Limestone and Groundwater:** The GDR/GBR should state that the upper limestone is fractured, and that groundwater will be present. **Flow** rates are difficult to assess; therefore, the contractor shall assume a tremie pour is required to seal the shaft base above bedrock.
- **Dewatering Limitations:** The GBR limits dewatering, with the intent that dewatering of the carbonate aquifer is not permitted. However, dewatering of overburden soils, silt seams, sand seams, existing trench beddings, and backfills that do not affect the aquifer are permitted.

3.3 Semple Outfall, Contract 4, Jefferson East CSR Project

The Semple Outfall Project was constructed as the outfall segment of the Semple Avenue Trunk Sewer and completed in 2016. The project was tunneled in glaciolacustrine clays similar to clays that will be encountered on sections of this Project. The project involved construction of 110 m of 2,100 mm diameter reinforced concrete pipe (RCP) at a depth of about 8 m. An Ackerman EBS840 with EX-50 Excavator shield was used for tunneling. Muck handling was with a conveyor and muck cars. Tunneling was completed in two drives of 40 and 70 m length. Pipe was jacked from the central launch shaft to the reception shafts. The shafts were constructed with temporary support consisting of soldier piles and timber lagging. There were no reported issues with shaft construction. Dewatering was not used for construction. The contractor experienced challenges with maintaining equipment in cold winter conditions and had difficulty in handling the cuttings of high plastic clays with the TBM belt conveyor. Additional details are provided in AECOM (2019).

General Lessons learned from the Northeast interceptor Sewer Project include the following:

- **TBM Oversteer/Overcorrection:** TBM oversteer/overcorrection should be avoided, as it significantly increases stress on jacking pipes.
- **Pipe Manufacture Monitoring:** Pipe manufacture should be carefully monitored. On this project, mismatched header pallets resulted in overstress on bell joints and caused several damaged pipes.

3.3.1 Jefferson East Combined Sewer Relief Works (Contract 5) Semple Avenue Trunk Sewer Project

The Semple Avenue trunk sewer project was an extension of the Jefferson East Combined Sewer Relief (CSR) Works, designed to upgrade the Jefferson East Combined Sewer District to meet the five-year level of service (LOS) design criteria. This project involved disconnecting surface runoff from the existing combined sewer system in the northern portion of the Jefferson district, thereby increasing capacity in the existing Jefferson Combined Sewer trunk. The Semple Avenue Trunk Sewer was tunneled in glaciolacustrine clays similar to the clays that will be encountered on sections of this Project and was completed in 2021. The tunnel was constructed as a 1,540 m single drive between the launch shaft and reception shaft A Lovat M-112 open face TBM with a 2845 mm diameter cutterhead was used to advance the tunnel The tunnel was constructed with a primary lining of steel ribs and timber lagging. Tunnel cuttings were handled by TBM conveyor and muck cars. Winter construction was involved. HOBAS - CCFRPM carrier pipe consisting of 400 m of 1,800 mm and 1,100 m of 2,100 mm was installed following completion of tunnelling. The carrier pipe was grouted in place. Grout loss during backfilling resulted in grout migrating into adjacent sewers.

General Lessons learned from the Northeast interceptor Sewer Project include the following:

- **Two-Pass Tunnel System:** The Jefferson project utilized a two-pass tunnel system (2.9 m diameter primary steel ring/timber lagging) with a 2100 mm GRP carrier. This system worked well, achieving a single drive of 1600 meters.
- **Annular Grout Breach:** An annular grout breach into the sewer system occurred, ultimately resulting in basements breached with annular grout. Lessons learned include:
  - The contractor needs to monitor and assess grout volume versus planned volumes.
  - The contractor needs to monitor grout pressure at discharge, not at the pump, to ensure pressures are within safe limits in the annulus.
  - The contractor needs to assess fill levels of staged grouting to ensure annular blockages between ports are not created, which could confine the grout.
  - The importance of establishing baseline vibration levels for construction monitoring, conducting preconstruction inspections of structures within the expected zone of influence and ensuring there is adequate means for the monitoring and control of grout volumes and grout loss.

3.4 Northwest Interceptor Sewer Extension

The Northwest Interceptor Sewer was installed in 2015 and 2016, within the lacustrine clay and silt till transition zone by pipe jacking using an Ackerman open face TBM. The project involved construction of about 1,600 m of 1350 mm diameter LDS pipe. Ground conditions encountered during tunneling included cobbles and boulders ranging in size up to 500 mm embedded within the lacustrine clay zone, as well as till undulations as the project moved west, also containing numerous boulders. Two rescue shafts were required during tunnel construction due to numerous cobble and boulder obstructions ahead of the TBM. A third rescue shaft was constructed for an alignment correction. A total of 13 shafts were used for the project and were constructed using either soldier pile and lagging or steel caissons for excavation support.

Lessons learned from this project include:

- **Tunneling Method and Boring Machine:** The selected tunneling method and boring machine must be matched to the expected ground conditions, including tunneling within the clay-till interface with concentrations of cobbles and boulders.

3.5 Trunk Sewer, Contract 4, Cockburn & Calrossie CSR Project

The Trunk Sewer and LDS Separation Project, Contract 4 was mined in glaciolacustrine clays similar to the clays that will be encountered on sections of this Project. The project was completed in 2017 and involved construction of about 525 m of 2,700 mm diameter Land Drainage Sewer (LDS) pipe at depths ranging from 8 to 8.5 m below grade. The project included two tunnel drives consisting of a 120 m drive under CN rail tracks and a 410 m drive from the launch shaft to Taylor Avenue. The contractor used a Herrenknecht AVN 2500 slurry MTBM to mine the tunnel. The TBM shield was increased (up skinned) to 2750 mm for the project. The contractor successfully used two centrifuges with the slurry treatment plant for separating clay cuttings from the slurry. The contractor was required to meet strict settlement criteria for the segment crossing under the CN rail right of way. Surface settlement was monitored to confirm compliance with the established limits. Results from monitoring prior to crossing under the CN ROW showed settlement had exceeded allowable levels. Tunneling under CN met the allowable settlement limits using a combination of maintaining TBM face pressure throughout the drive and injection of bentonite grout through ports in the RCP. Three circular self-sinking shafts were constructed, one launch shaft and two retrieval shafts. The self-sinking method used a surface form to cast the concrete lining and a sacrificial sinking shoe. The shaft lining dropped under self-weight as the interior of the shaft was excavated. Construction vibrations were not reported as an issue. The launch shaft incorporated the alignment deflection of the two drives. Additional details on this project are provided in Fordyce (2018), AECOM (2018 and 2019), KGS (2016 and 2019) and Trek 2025.

3.6 Taylor Avenue Trunk, Contract 5, Cockburn & Calrossie CSR Project

The Taylor Avenue Trunk, Contract 5, was mined in glaciolacustrine clays similar to the clays that will be encountered on sections of this Project. The project involved construction of about 700 mm, 2,100 mm and 2,400-mm diameter fiberglass LDS pipe and was completed in 2020. The tunnel alignment was located below and close to multiple utilities including transmission towers, gas and water mains, sewers and communications lines. These constraints resulted in an alignment with vertical and horizontal curves and restrictions on locations for intermediate shafts. Tight settlement criteria were established to limit impact on adjacent utilities. The contractor used a 3,335 mm diameter Lovat open-face TBM equipped with pressure relieving gate and flood doors. The TBM cutterhead was equipped for tunneling in clays with a high clogging potential. The TBM incorporated an articulated steering shield to meet the vertical and horizontal curve requirements of the alignment. Handling of tunnel cuttings was with rail and muck cars. The tunnel was constructed as a single drive between the launch shaft and the retrieval shaft with a primary lining

consisting of steel ribs and timber lagging installed as the tunnel was advanced. The final fiberglass LDS pipe was installed and grouted in place following completion of mining and installation of stub-outs for future connections. Additional details on this project are provided in Fordyce (2018), AECOM (2019), KGS (2019) and Trek 2025.

Key lessons learned from this project included:

- **Tunnel Completion:** Successful completion of a tunnel in a constrained alignment and high plastic clay using ribs and lagging as the primary lining.
- **Settlement Monitoring Program:** Effective use of a settlement monitoring program for control of settlement and limiting impact on nearby sensitive infrastructure.

4. Geological Setting

4.1 Regional Geology

In general, the soils encountered during the investigation consisted of fill underlain by fat clay. The regional geology of the site has been outlined in the AECOM (April 2025) Geotechnical Data Report (GDR) and should be reference in conjunction with Section 4 of this Report for a more detailed outline of the regional geological setting.

Site-specific geotechnical and geological information derived from the AECOM 2024 geotechnical investigation and past investigations (including results of the geotechnical drilling and laboratory test data) are also presented in the GDR. The full GDR can be found in **Appendix I.**

4.2 Topography

The topography along the FGSV Siphon alignment varies significantly as the site is located at a river crossing. The elevation along the eastern riverbank varies between approximately 230 m above sea level (mASL) and 235 m ASL at its crest and decreases sharply towards the centre of the river channel to an approximate elevation of 218 m. The ground surface along the crest of the western riverbank varies between 227 m ASL and 238 m ASL and, in turn, falls sharply to the centreline of the river channel. The proposed excavation work involves constructing a 10 m diameter base shaft at the launch and receiving site, located on the east and west side of the riverbank slope. It is understood that this work will not impact the existing riverbank profiles, as the siphon chambers are situated away from the riverbank slopes.

Any plans to disturb the riverbank slopes should be submitted to the Consultant for review prior to construction. The ground surface profile along the sewer alignment is shown on the **General Plan** within the **Contract Documents and Drawings.**

5. Summary of Subsurface Investigation

As described in the AECOM (April 2025) GDR, AECOM conducted a geotechnical investigation in 2024 along the proposed FGSV Siphon alignment with the objective of characterizing the subsurface ground and groundwater conditions along the new alignment. The findings of the AECOM 2024 geotechnical investigation, including groundwater level readings in 2025, are summarized in the GDR found in **Appendix I**, with the pertinent findings of the investigations are also presented below.

5.1 Previous Geotechnical Investigations

5.1.1 Geotechnical Condition Assessment (AECOM, 2021)

A previous geotechnical investigation completed near the project site has also been referenced within the AECOM (April 2025) GDR. This previous geotechnical investigation that was referenced within the GDR was carried out to support condition assessment of the FGSV Siphon Crossings, found between the two Fort Garry Bridges, just north of the proposed FGSV siphon alignment. The historical geotechnical information has also been summarized in the following sections of this report. AECOM reviewed these previous geotechnical investigations as part of our abandonment/siphon works at FGSV.

As described in the project GDR, a geotechnical condition assessment was conducted by AECOM in 2021 for the FGSV Siphon Crossing. The geotechnical condition assessment for the existing Fort Garry Siphon Crossings, involved reviewing available background information and conducting a visual field inspection within a 30-meter zone around the crossing. The assessment aimed to evaluate potential risks of slope instability and erosion affecting the buried sewer and water systems. The findings from the review and inspection were used to assign Slope Condition Grade (SCG) and Erosion Condition Grade (ECG), helping to determine the need for further geotechnical investigation or slope stability Analysis. Detailed information of AECOM's geotechnical condition assessment (AECOM 2021) is provided in **Appendix 1** of the GDR.

5.1.2 Geotechnical Assessment Ft. Garry-St. Vital Feeder Main (AECOM, 2018)

AECOM conducted a geotechnical assessment of the Ft Garry-St Vital Feeder Main in 2018 as part of a condition assessment of the feeder main. The feeder main is located between the twin Ft Garry bridges immediately north of the existing sanitary sewer siphons. Results of that geotechnical condition assessment was that the west bank global stability between the bridges was slightly less that the desired factor of safety of 1.5 for critical infrastructure.

5.1.3 Geotechnical Investigation for Ft Garry Bridges (Klohn Leonoff, 1976)

The geotechnical assessments included within the appendix are testhole logs in support of the Fort Garry Bridge construction by Klohn Leonoff Consultants Ltd in1975/76. This comprised of eleven testholes for the south bridge and eight (8) testhole logs for the north bridge. AECOM does not have access to the full geotechnical report for the testholes. A summary of the drilling and testing components are shown in the tables below.

Table 5-1: Summary of Testholes for the South Bridge

Testholes	Testhole Elevation (mASL)	Location	Completion Depth (m)	Bedrock Contact Elevation (mASL)	Completion Elevation (mASL)	Stratum
TH 1004	230.429	Western Riverbank	15.85	216.865	214.579	Bedrock
TH 401	228.905		12.19	n/a	216.715	Till
TH 1	227.442		14.66	216.469	212.782	Bedrock
TH 11	221.712	Riverbed	11.80	216.225	209.912	Bedrock
TH 12	221.712		10.45	216.173	211.262	Bedrock
TH 6	221.742		9.54	216.713	212.202	Bedrock
TH 5	221.742		10.67	216.509	211.074	Bedrock
TH 4	226.863	Eastern Riverbank	14.54	217.313	212.324	Bedrock
TH 1002	229.514		16.15	216.408	213.364	Bedrock
TH 402	229.667		13.72	n/a	215.947	Till
TH 403	231.191		14.63	n/a	216.560	Till

Table 5-2: Summary of Testholes for the North Bridge

Testholes	Testhole Elevation (mASL)	Location	Completion Depth (m)	Bedrock Contact Elevation (mASL)	Completion Elevation (mASL)	Stratum
TH 1003	230.429	Western Riverbank	16.76	216.408	213.665	Bedrock
TH 2	227.076		16.06	216.256	211.013	Bedrock
TH 9	221.681	Riverbed	9.75	216.499	211.927	Bedrock
TH 10	221.681	Riverbed	10.27	216.499	211.409	Bedrock
TH 8	221.742		13.08	216.332	208.666	Bedrock
TH 7	221.742		10.97	216.332	210.769	Bedrock
TH 3	227.106	Eastern Riverbank	15.48	216.338	211.623	Bedrock
TH 1001	231.648		18.29	216.408	213.360	Bedrock

5.2 AECOM 2024 Geotechnical Investigation

From June 3 to August 9, 2024, five (5) test holes (TH24-01 to TH24-05) were drilled at the approximate locations shown in **Appendix 2** within the GDR. Test holes TH24-01 and TH24-02 were drilled along the west embankment in the vicinity of the west shaft location, test hole TH24-03 was drilled within the Red River channel, and test holes TH24-04 and TH24-05 were drilled on the east embankment in the vicinity of the east shaft location.

Drilling was completed by Paddock Drilling using the following equipment: track-mounted Acker Renegade drill rig equipped with 125 mm solid stem augers and HQ-sized (96 mm OD) core barrel for test holes TH24-01, TH24-02, TH24-04 and TH24-05, and Cricket B20 equipped with BQ sized (60 mm OD) core barrel mounted on a floating barge for test hole TH24-03. Subsurface conditions observed during drilling were visually classified and documented by AECOM geotechnical personnel. Other pertinent information, such as groundwater and drilling conditions, were also recorded during the field investigation.

Disturbed soil samples collected from auger cuttings and split-spoon samplers, as well as relatively undisturbed Shelby Tube samples, were obtained at regular intervals. Standard penetration tests (SPTs) were completed at selected intervals in the test holes, and blow counts for 300 mm penetration (SPT "N" blow counts) were recorded. NQ and HQ rock core samples were logged in the field and collected for further analysis. Recovered soil and rock core samples were transported to AECOM's materials testing laboratory in Winnipeg for further visual examination and testing.

The bedrock cores were logged at AECOM's materials testing laboratory, recording the type of bedrock, Total Core Recovery (TCR), Solid Core Recovery (SCR) and Rock Quality Designation (RQD).

Monitoring wells (50 mm diameter PVC pipes) were installed in two test holes (TH24-01 and TH24-05) to measure groundwater depths. The test hole logs, and groundwater instrumentation details and measurements are provided in the GDR.

5.3 Laboratory Testing

Soil samples collected during the geotechnical investigations were tested at Geomechanica's Materials Testing Laboratory in Oakville, Ontario, and AECOM's Materials Testing Laboratories in Winnipeg, Manitoba for soil classification and estimation of engineering properties. The bedrock core samples were tested in Eng-Tech Consulting Ltd., Laboratories in Winnipeg, Manitoba to estimate uniaxial compressive strength (UCS). Details of the type and number of tests are presented in **Table 5-3**. The laboratory test results for test holes drilled along the FGSV Siphon alignment are provided in the GDR.

Table 5-3: Laboratory Testing (AECOM 2024 Geotechnical Investigation)

Laboratory Testing	Number of Tests Completed
Moisture Content	60
Particles Size Analysis (Hydrometer Analysis)	15
Atterberg Limits	15
Unconfined Compressive Strength (Soil)	10
Unconfined Compressive Strength of Intact Rock Core	5
Abrasiveness of Rock Using the CERCHAR Abrasiveness Index Method	5

6. Ground Characterization

6.1 General Stratigraphy

The subsurface stratigraphy along the FGSV Siphon alignment generally comprises of mixed alluvial soils (sand, silt and clay) overlying (in descending order) glacio-lacustrine clay, glacial till deposits (sand and silt till), and carbonate bedrock (predominately limestone and dolomitic limestone). The bedrock surface was typically encountered at an elevation of between 217.21 m and 215.78 m. The composition of the alluvial soils is expected to vary with depth and between riverbanks (and at the proposed siphon outfall chamber locations). Cobbles and boulders should be expected within the glacial till deposit (typical of glacial till soils within the Winnipeg area).

Detailed descriptions of the subsurface conditions encountered at the testholes locations are shown on the test holes logs in Appendix 3 and Appendix 4 of the GDR. A brief description of the subsurface soil/bedrock units encountered along the FGSV Siphon alignment, and their engineering properties is provided in the following Sections.

6.2 Subsurface Profile

The soil stratigraphy on the project site generally consists of topsoil, clay fill overlying a clay deposit, which is underlain by silt till and bedrock. Additionally, alluvial deposits were observed at the riverbank and along the river bottom. Detailed descriptions of the strata and related field and laboratory data are provided in Sections 5 and 7 of the GDR.

6.2.1 Topsoil

Topsoil was encountered at the ground surface in testholes TH24-01, TH24-02, TH24-04, and TH24-05. The thickness of the topsoil was approximately 0.30 m and is observed to be black, moist, with organic content, with traces of sand, gravel, and silt. The moisture content of the topsoil ranged from 31.4% to 35.6%.

6.2.2 Fill – Clay (CH)

Fat clay (CH) fill material was encountered in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from approximately 0.7 m to 1.9 m. The fat clay (CH) fill layer was generally observed to be moist, of high plasticity, black in color, firm to stiff and have traces of sand, gravel, and silt. The moisture content of the fat clay (CH) fill ranged from 32.8% to 35.6%.

6.2.3 Clay (CH)

Grey fat clay (CH) was encountered below the clay fill in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from 10.10 m to 15.75 m. It is observed to be moist, firm, and of high plasticity with trace of silt. The clay shear strength varies from firm to soft and decreases with depth. The moisture content of the fat clay (CH) ranged from 13.6% to 51.3%.

6.2.4 Silt (ML) Till

Tan silt (ML) till was encountered below the fat clay material in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from 0.71 m to 1.95 m. It is observed to be moist, loose, and of low plasticity with trace of sand, and clay and gravel. The silt (ML) till was compact with moisture content of the silt (ML) till ranged from 11.4% to 18.5%. Cobbles and boulders should be expected within the glacial till deposit (typical of glacial till soils within the Winnipeg area).

6.2.5 Bedrock

Bedrock (BR) was encountered below the silt (ML) till in the cored testholes TH24-01, TH24-03 and TH24-05. Brecciated Dolomitic Mudstone was the type of rock observed in the coring, a Lower Fort Garry Member of the Red River Formation. The Brecciated Dolomitic Mudstone was observed at elevations of 216.38 m ASL and 217.20 m ASL to beyond 207.20 m ASL and 182.53 m ASL. The dolomitic limestone was white greyish and was nodular bedded.

6.3 Bedrock Characterization

6.3.1 General

Most of the bedrock encountered at the site, specifically along the proposed FGSV Siphon alignment, consists of Brecciated Dolomitic Mudstone. The bedrock surface elevation varied between 217.21 mASL and 215.78 mASL along the proposed FGSV Siphon alignment. The bedrock is generally white greyish, medium strong to very strong. The bedrock units encountered are consistent with geological maps of the area. Details of bedrock UCS, RQD, SCR and RQD are provided in Section 7 of the GDR.

6.3.2 Rock quality Designation (RQD)

RQD ranges from 0% to 94% which represents very poor to excellent quality bedrock. Lower RQD values were typically found at depths closer to the bedrock surface, but RQD values are typically consistent between an approximate elevation of 215.24 mASL to 187.10 mASL. RQD values at each test hole location are shown in Section 7.1.4 of the GDR.

6.3.3 Unconfined Compressive Strength of Intact Rock Core Specimens

Unconfined Compressive Strength of Intact Rock Core Specimen testing was performed on samples of Brecciated Dolomitic Mudstone from the Red River Formation. The Brecciated Dolomitic Mudstone is classified as medium strong to very strong. The measured unconfined compressive strength of the intact rock for the Brecciated Dolomitic Mudstone range between 35.3 MPa and 128.0 MPa. More details regarding the Unconfined Compressive Strength of Intact Rock Core Specimen are found within **Appendix I**.

6.3.4 Bedrock Permeability

High permeability zones could be encountered at various bedrock contacts and within the upper bedrock near the ground surface, approximately 5 mBGS (216.5 mASL). The MTBM operating in closed-face is slurry-supported using a bentonite suspension drilling fluid. The slurry pump and face pressure should be monitored to ensure excessive pressure is not applied to the tunnel face. These zones of high permeability may provide preferential pathways for drilling fluid and annular lubrication fluid flow, depending on the features contributing to the high permeability. These features can include, but are not limited to, fracture networks, joint networks, shear zones, or areas of weathered rock.

6.4 Groundwater and Sloughing Conditions

Groundwater seepage or soil sloughing conditions were observed in most testholes upon completion of drilling. Details of the location and nature of the sloughing, seepage, and groundwater encountered are provided in Section 6 of the GDR, as well as in the testhole logs in **Appendix 3** of the GDR

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

6.4.1 Site Specific Groundwater Observations

Groundwater elevations were measured in the test holes during and after the completion of AECOM geotechnical investigation. The measured groundwater levels are also presented in Section 6.1 **(Table 6)** of the GDR.

Groundwater instrumentation along the FGSV Siphon alignment consists of two (2) standpipe piezometers installed as part of the AECOM 2024 geotechnical investigations. Instrumentation was installed into the bedrock along the FGSV Siphon alignment, and the instruments were monitored between June 4, 2024, and March 12, 2025, by AECOM.

A graphical summary of these results is provided in **Figure 6-1** and can also be found in the GDR**.**

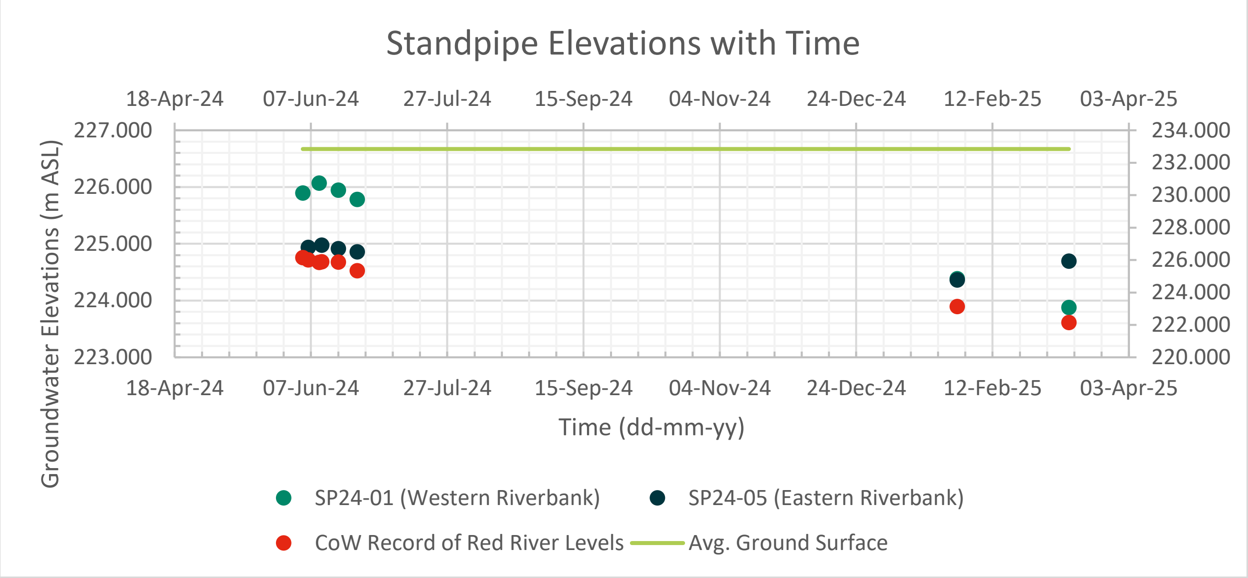

Figure 6-1: Graph of Groundwater Elevations Versus Time

It is anticipated that the launch and receiving pit will be constructed mostly within fat clay, at a depth of approximately 15 mBGS (216.40 mASL) and 10 mBGS (222.7 mASL). The typical range of hydraulic conductivity for fat clay is between 1x10^{-10} to 1x10^{-6} cm/s. Thus, there will be no significant groundwater seepage expected from within the fat clay. For the daily water level for Red River at James Avenue Pumping Station, see **Appendix III**.

6.4.2 Groundwater Dewatering Rates

Drawdown of the aquifer is not permitted to facilitate shaft construction as part of the project. Therefore, the Contractor is required to use 'sealed' methods for shaft construction. Watertight shafts must include a sealed concrete base designed to prevent basal heave, resist hydrostatic pressures, and minimize the ingress of fines and groundwater infiltration. Additionally, it is recommended that the Contractor does a pumping test prior to construction for the entry shaft, where bedrock is found at the bottom of the excavation. This pumping test is meant to facilitate the nuisance dewatering of the launch shaft and not the underlying aquifer.

The Contractor is responsible for conducting the necessary hydrogeological assessments and tests at each shaft location to determine appropriate dewatering rates.

7. Discussions and Recommendations

This section of the report presents geotechnical engineering insights regarding the proposed installation of the casing and carrier pipe, which will run parallel to the Fort Garry Bridge and cross the Red River to the south of the bridge.

The following geotechnical input is based on the information available at the time of this report. Comments regarding construction are included to highlight aspects that may impact the design. Contractors should review the factual results of the investigation to ensure the adequacy of the information for construction. They must interpret the data concerning the profile provided during the tendering phase, as this will influence their construction techniques, schedule, safety measures, and equipment capabilities.

7.1 Soil and Bedrock Stratigraphic Summary

The stratigraphic summary shown below has been developed in consideration of the conditions encountered in the testholes. A more detailed version is found in Section 5 of the GDR.

- Topsoil: black in colour, moist, with organic content, and with trace of sand, gravel and silt.
- Clay (CH) Fill: black in colour, moist, high plasticity, firm to stiff and have traces of sand, gravel and silt.
- Clay (CH): grey in colour, moist, firm, and of high plasticity with trace of silt.
- Silt (ML) Till: tan in colour, loose, and of low plasticity with trace of sand, clay, and gravel.
- Bedrock: grey to dark grey in colour and was nodular bedded.

The lowest and highest groundwater levels recorded at the monitoring wells TH24-01 and TH24-05 were at elevations of **223.874 mASL** and **224.754 mASL**, respectively. These readings, taken from June 4, 2024, to March 12, 2025, indicate that both the bottom of the shaft and the tunnel are located below the water table. The anticipated elevations of the shaft at the launch and receiving shafts are **216.40 mASL** and **222.70 mASL**, respectively.

7.2 Anticipated Ground Behaviour

7.2.1 Overburden

For the description of the anticipated behaviour of the overburden deposits, the Tunnelman Ground Classification System, developed by Terzaghi (1950) and modified by Heuer (1974), has been adopted. It should be noted that the Tunnelman ground classification terms provide a description of the behaviour of the different soil types at an unsupported vertical tunnel face under atmospheric conditions. As the tunnelling is to be constructed using MTBM, the Tunnelman descriptions have only been provided to give a general idea of soil face stability behaviour.

The baseline behaviour of the overburden soil units is presented in **Table 7-1.**

Table 7-1: Anticipated Behaviour of Soil at Unsupported Vertical Tunnel/ Excavation Face

Soil Group	Soil Type and Description	Anticipated Behaviour
Alluvial Cohesive Soil Unit	Clayey Silt, Silty Clay	Will be stable and exhibit firm behaviour initially after excavation but depending on the degree of fissuring will degrade into Slow Raveling ground both above and below the groundwater table. 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 should be covered with waterproof material to prevent saturation of the soil and all surface runoff to be directed away from the excavations.
Glacio-Lacustrine Clay (Cohesive) Unit	Silty Clay	The upper layer of the glacio-lacustrine clay will be stable and exhibit firm behaviour initial upon excavation and quickly in-turn become Slow Raveling depending upon the degree of fissuring. The lower layer will begin to Squeeze and yield plastically with increased depth upon excavation. The shear strength of both the upper and lower silty clay will progressively decrease over a short period of time due to changes in effective stress and moisture conditions, resulting in Swelling and yielding conditions of the soil if left unsupported.
Alluvial Granular Soil Unit	Sand, Sand and Gravel	Above the groundwater table these soil types will be Fast Raveling or exhibit cohesive running but will immediately Flow below the groundwater table even under a small groundwater head (< 1 m).
Glacial Till (Granular)	Sandy Silt, Silty Sand	Below the groundwater table, Fast Raveling to Flowing conditions will occur. Unstable (Running or Flowing) conditions can be expected where cohesionless granular layers or pockets are present in the till. Cobbles and boulders will be encountered.

These baseline conditions should be considered during the planning and execution of the tunnelling project to ensure stability and safety.

7.2.2 Clogging Potential

Clogging potential refers to the likelihood of soil particles adhering to the cutting tools and conveyor systems of Microtunneling Boring Machines (MTBM). This stickiness can lead to clogging and blockages and can significantly impact the efficiency and safety of tunneling operations, making it crucial to assess and mitigate in advance. The methodology for assessing clogging potential was developed by F. Hollman and M. Thewes in 2013, focusing on evaluating soil properties such as plastic limit (PL), liquid limit (LL) and moisture content to predict and manage clogging risks.

The consistency and behaviour of soil containing clay minerals can change with interaction of tunneling equipment and water and conditioning agents (i.e. groundwater seepage or construction water). Based on the clogging charts that show groups of data points using the results from Atterberg's Limits testing on samples of the fat clay and till materials, our analysis shows strong to medium clogging for the fat clay layers, and little clogging for the glacial till.

It is the contractor's responsibility to carry out compatibility tests with different conditioning agents, dosing levels, and moisture contents so that field operation prevent clogging from occurring. Details regarding PL, LL and moisture content is found within the GDR and shall be considered by the Contractor for clogging potentials before construction commences.

7.2.3 Bedrock

This section describes the anticipated behaviour of the bedrock at an unsupported vertical tunnel face under atmospheric conditions. The following description will apply to sections of shafts in bedrock and will also give a general idea of face stability behaviour in the tunnel sections where bedrock is encountered.

Wedge-shaped blocks will be released and fall into the tunnel excavation under the following conditions:

i. where nearly vertical joint sets intersect the tunnel at a shallow angle in combination with bedding planes and/or weak horizontal seams; and,
ii. where horizontal bedding planes intersect two inclined joints. This type of wedge instability is expected to occur on a localized basis and can be expected to occur at any time following tunnel excavation.

Roof slab fallout can occur in the bedrock where a clay-filled or open, weak horizontal seam is present in the tunnel crown. This type of fallout occurs along the tunnel until the weak seam pinches out or rises sufficiently above the crown.

Table 7-2: Anticipated Behaviour of Bedrock at Unsupported Vertical Tunnel/ Excavation Face

Layer	Type and Description	Anticipated Behaviour
Bedrock	Lower Fort Garry Member of the Red River Formation: Brecciated Dolomitic Mudstone	The un-weathered competent bedrock units will be stable and Firm upon excavation. Fast Raveling conditions will be encountered depending upon the degree of rock fracturing and discontinuities within the bedrock formation.

7.3 Hydrogeological Investigation

If required, it is the contractor's responsibility to conduct a hydrogeological investigation to manage the groundwater, which would allow for deep excavations at the project (as well as at locations within the tunnel). The hydrogeological investigation will need to include, but is not limited to:

- Test well drilling
- Aquifer pump testing
- Technical analysis

7.4 Recommended Geotechnical Baseline Parameters

7.4.1 Launch Shaft (East Riverbank) Geotechnical Baseline

The proposed bottom elevation of the launch shaft is 216.40 mASL, on top of bedrock. **Table 7-3** and **Table 7-4** summarize the baseline parameters for the launch shaft, located near testhole TH24-05.

Table 7-3: Subsurface Profile and Baseline Parameters for Launch Shaft

Subsurface Profile	Elevation	Baseline Parameter
Fill – Clay (CH)	231.63 m	Thickness = 0.61 m
Brown Clay (CH)	231.02 m	Thickness = 6.72 m USS[1] = 50 kPa Unit Weight = 19 kN/m^3 Effective Cohesion = 3 kPa Effective Angle of Internal Friction = 20 degrees Plastic Limit = 20% Liquid Limit = 90% Plastic Index = 70 Moisture Content = 28% Liquid Index = 0.11
Grey Clay (CH)	224.30 m	Thickness = 5.18 m USS[1] = 25 kPa Unit Weight = 19 kN/m^3 Effective Cohesion = 3 kPa Effective Angle of Internal Friction = 20 degrees Plastic Limit = 20% Liquid Limit = 90% Plastic Index =70% Moisture Content = 40% Liquid Index = 0.20
Silt (ML) Till	219.12 m	Thickness = 1.92 m Relative Density = Dense Unit Weight = 20 kN/m^3 Angle of Friction = 35 degrees SPT N Value = 50 % Gravel = 8 % Sand = 55 % Fines = 37 Moisture Content = 15.5% It is anticipated that boulders less than 1 m^3 in size will be encountered.
Bedrock	217.20 m	Lithology = Lower Red River Formation: Dolomitic Mudstone, Brecciated UCS[2] = 125 MPa (ISRM Classification: Very Strong) CAI[3] = 1.6 (ASTM Classification: Medium) RQD[4] = 45%
Basal Instability		
Launch Shaft	216.40	Since the bottom of the excavation is found on the bedrock, excavation base stability is not a concern.
Buoyancy Uplift from Excess Groundwater Pressure Beneath an Impermeable Stratum		
Launch Shaft	216.40	Since the bottom of the excavation is found on the bedrock, it is not applicable. The contractor should develop a plan to manage artesian pressures. A professional engineer specializing in excavation design should be consulted before construction begins.

[1] *USS = Undrained Shear Strength*
[2] *USC = Unconfined Compressive Strength*
[3] *CAI = CERCHAR-Abrasivity-Index of the sample that is calculated by taking the mean wear and multiplying it by 10*
[4] *RQD = Rock Quality Designation (International Society of Rock Mechanics (ISRM) Standard, 1979)*

Given the potential for seasonal fluctuation of the groundwater table, it is recommended that the groundwater level in the SP's be measured again prior to construction to confirm any change arising from seasonal variation or changed conditions since the time of previous monitoring event. As a baseline, the table below shows the recommended groundwater and river levels to be utilized for each season.

Table 7-4: Seasonal Groundwater Levels for Launch Shaft

Location	Piezometer ID	Season	GW Reading	Historical River Levels
East Riverbank	SP24-05	Spring Summer[1]	Highly Variable ~227.718 mASL	Highly Variable 223.98 mASL
		Winter	~224.75 mASL	221.76 mASL

[1] *Based on Daily Water Level Graph (see* ***Appendix III****.)*

7.4.2 Riverbed Tunnel Geotechnical Baseline

As previously mentioned, the lowest point of pipe is within the river and will be tunneled through bedrock at an invert elevation 207 m. **Table 7-5** summarizes the baseline parameters for the bedrock at this location which is located near testhole TH24-03.

Table 7-5: Subsurface Profile and Baseline Parameters for Riverbed Tunnel

Subsurface Profile	Elevation	Baseline Parameter
Red River	Spring = Highly Variable Summer[4] = 223.98 mASL Winter = 221.76 mASL	• High Variability in Spring • Summer levels. Controlled by St. Andrews Lock and Dam • Small variability in Winter
Silt (ML) Till	217.60 mASL	Thickness = 1.92 m Relative Density = Dense Unit Weight = 20 kN/m^3 Angle of Friction = 35 degrees SPT N Value = 50 blows/300 mm penetration % Gravel = 8 % Sand = 55 % Fines = 37 Moisture Content = 15.5% It is anticipated that boulders less than 1 m^3 in size will be encountered.
Bedrock	215.80 mASL	Lithology = Lower Red River Formation: Dolomitic Mudstone, Brecciated UCS1 = 164 MPa (ISRM Classification: Very Strong) CAI2 = 1.6 (ASTM Classicisation: Medium) RQD[3] = 47%

[1] *USC = Unconfined Compressive Strength.*
[2] *CAI = CERCHAR-Abrasivity-Index of the sample that is calculated by taking the mean wear and multiplying it by 10.*
[3] *RQD = Rock Quality Designation (International Society of Rock Mechanics (ISRM) Standard, 1979).*
[4] *Based on Daily Water Level Graph (see* ***Appendix III****).*

7.4.3 Receiving Shaft (West Riverbank) Geotechnical Baseline

The proposed bottom elevation of the receiving shaft is 222.7 m within fat clay. **Table 7-6** and **Table 7-7** summarize the baseline parameters for the launch shaft, located near testhole TH24-01.

Table 7-6: Subsurface Profile and Baseline Parameters for Receiving Shaft

Subsurface Profile	Elevation	Baseline Parameter
Fill – Clay (CH)	233.50 mASL	Thickness = 0.45 m
Brown Clay (CH)	233.05 mASL	Thickness = 11.45 m USS[1] = 50 kPa Unit Weight = 19 kN/m^3 Effective Cohesion = 3 kPa Effective Angle of Internal Friction = 20 degrees Brown Clay USS[1] = 50 kPa Plastic Limit = 20% Liquid Limit = 90% Plastic Index = 70% Moisture Content = 38% Liquidity Index = 0.26
Grey Clay (CH)	221.60 mASL	Thickness = 4.26 m USS[1] = 25 kPa Unit Weight = 19 kN/m^3 Effective Cohesion = 3 kPa Effective Angle of Internal Friction = 20 degrees Grey Clay USS[1] = 25 kPa Plastic Limit = 20% Liquid Limit = 90% Plastic Index = 70% Moisture Content = 49% Liquidity Index = 0.29
Silt (ML) Till	217.30 m	Thickness = 0.75 m Relative Density = Dense Unit Weight = 20 kN/m^3 Angle of Friction = 35 degrees SPT N Value = 50 per 300 mm penetration % Gravel = 10.4 % Sand = 33.5 % Fines = 56.1 Moisture Content = 13.8 % It is anticipated that boulders less than 1 m^3 in size will be encountered.
Bedrock	216.80 mASL	Lithology = Lower Red River Formation: Dolomitic Mudstone, Brecciated UCS[2] = 125 MPa (ISRM Classification: Very Strong) CAI[3] = 1.6 (ASTM Classicisation: Medium) RQD[4] = 56%
Basal Instability		
Receiving Shaft	222.70 mASL	As per Section 20.8.2.1 of the CFEM, base heave is deemed satisfactory if (FS) heave is greater than 1.5. The design of the shoring should be carried out by a professional engineer specialized in shoring design with a baseline value of (FS)heave of 1.5 or greater.
Buoyancy Uplift from Excess Groundwater Pressure Beneath an Impermeable Stratum		
Receiving Shaft	222.0 mASL	As per Section 22.3.1 of the CFEM, buoyancy uplift due to excess groundwater pressure beneath an impermeable stratum is deemed satisfactory if FS is greater than 1.1. The contractor should develop a plan to manage artesian pressures.

[1] *USS = Undrained Shear Strength*
[2] *USC = Unconfined Compressive Strength*
[3] *CAI = CERCHAR-Abrasivity-Index of the sample that is calculated by taking the mean wear and multiplying it by 10*
[4] *RQD = Rock Quality Designation (International Society of Rock Mechanics (ISRM) Standard, 1979)*

Given the potential for seasonal fluctuation of the groundwater table, it is recommended that the groundwater level in the SP's be measured again prior to construction to confirm any change arising from seasonal variation or changed conditions since the time of previous monitoring event. As a baseline, the table below shows the recommended groundwater levels to be measured for each season.

Table 7-7: Seasonal Groundwater Levels for Receiving Shaft

Location	Piezometer ID	Season	GW Reading	Historical River Levels
West Riverbank	SP24-01	Spring	Highly Variable	Highly Variable
		Summer[1]	~225.921 mASL	223.98 mASL
		Winter	~224.384 mASL	221.76 mASL

[1] *Based on Daily Water Level Graph (see* ***Appendix III****)*

7.5 Tunnelman's Ground Classification and Probable Working Conditions

Table 7-8 is included for completeness and general reference. This table outlines the framework for Tunnelman's Ground Classification and details the corresponding tunnel working conditions, as described by Heur and Virgins (1987), Brandt (1970), and others.

Table 7-8: Tunnelman's Ground Classification and Probable Work Conditions

Classification	Representative Soil Types	Tunnel Working Conditions
Hard	Very hard calcareous clay; cemented sand and gravel	Tunnel heading may be advanced without roof support
Firm	Loss above GWT; Various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support, and the permanent support can be constructed before the ground will start to move
Slow Raveling and Fast Raveling	*Fast Raveling* occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be *Slowly Raveling* or even Firm	Chunks or flakes of material begin to drop out of roof or the sides sometime after the ground has been exposed.
		In *Fast Raveling* ground, the process starts within a few minutes; otherwise, it is classed as *Slow Raveling*
Squeezing	Soft or medium-soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel (may not be noticed in tunnel but cause surface subsidence)
Swelling	Heavily pre-compressed clays with a plasticity index more than about 30; Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movements are associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	*Cohesive running* occurs in clean, fine moist sand *Running* occurs in clean, coarse or medium sand above the GWT	The removal of the lateral support of any surface rising at an angle of more than about 34^{o} to the horizontal is followed by a 'run,' whereby the material flows like granulated sugar until the slope angle becomes equal to about 34^{o}. If the 'run' is preceded by a brief period of raveling, the ground is called *Cohesive Running*
Very Soft Squeezing	Clays and silts with high plasticity index	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size more than about 0.005 mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides but also through the bottom. If the flow is not stopped, it continues until the tunnel is completely filled.
Bouldery	Boulder glacial till; rip-rap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay or combinations of thereof.	Problems occurred in advancing shield or fore poling; blasting or hand mining ahead machine may become necessary.

For reference, stiff to firm fat lay below the groundwater level is anticipated to exhibit a 'slow raveling' to 'squeezing' behaviour.

7.6 Geotechnical-Based Assessment for MTBM

7.6.1 Micro-Tunneling Boring Machine (MTBM)

It is understood that the preferred method of installation of the siphon is by pipe jacking with MTBM. In general, the siphon is constructed by consecutively pushing pipes and the tunneling machine through the ground, using a jacking system for thrust. MTBM's are used with a mechanized excavating equipment that is remotely controlled, steerable, guided and articulated, connected to, and jacked forward by the pipe being installed. A tunneling machine has a rotating cutterhead that rotates and excavates the soil which comes inside the cutting head. The spoil is transferred to the rear of the shield via slurry lines or through conveyers which dump it into muck carts and conveys it out of the tunnel through the pipe being installed. Thrust power of hydraulic jacks is utilized to force the tunneling machine and the following string of pipes forward. The hydraulic pressures overcome face resistance and friction forces on the exposed surfaces of the shield and installed pipes.

It is understood that the installation of the pipes will be through a tunneling machine based upon local availability and expertise. Systematic settlements (typically small) and other operational settlements can occur when pipe jacking with tunneling machine is used.

When used with pipe jacking techniques, tunneling machines can advance pipelines several hundred metres to very accurate tolerances. Tunneling machines can be used in varying ground conditions, and high-water tables.

7.6.2 Installation Risks

Pipe Jacking with Tunneling Machine for the FGSV siphon has been evaluated against the following perceived risks:

- Ground settlement
- Buried Obstructions
- Clogging Potential
- Void Development
- Bedrock Considerations
- Groundwater
- Pipe Alignment and Grade Control

7.6.2.1 Ground Settlement

Major settlement is not anticipated on existing road embankments and riverbanks due to the use of Micro Tunnel Boring Machines (MTBM). Most of the siphon is submerged, mitigating settlement concerns. The riverbanks consist mainly of grass areas, further reducing settlement risks. However, the siphon may pass beneath a bicycle path, which will require monitoring.

Moderate to heavy groundwater seepage was recorded in test holes TH24-01, TH24-02, TH24-04, and TH24-05, within a clay layer at depths of 6.1 m (Elev. 225.1 mASL) to 10.4 m (Elev. 220.8 mASL). Soil sloughing was observed in test holes TH24-01, TH24-02, and TH24-03 at depths of 9.1 m (Elev. 222.1 mASL) to 16.5 m (Elev. 214.7 mASL).

While pipe jacking minimizes ground disturbance, small settlements can occur due to:

- **Systematic Settlement**: Resulting from the collapse of the overcut between the excavation and the trailing pipeline (E.g. Annular Collapse).
- **Operational Settlements**: Caused by over-excavation due to operator inexperience or unexpected ground conditions.

To mitigate these risks, lubricating slurry should be applied to fill annular voids, preventing collapse. This slurry can be replaced with cementitious grout upon completion. The contractor must ensure they have the necessary equipment for effective grouting and be prepared to address any instability at the tunnel face based on observed settlements.

7.6.2.2 Buried Obstructions

Buried obstructions were not encountered during AECOM'S geotechnical investigation in June and August 2024. Obstructions such as cobbles and boulders are likely in till interface and possible in lacustrine clays. Encountering buried obstructions can prevent or slow down the progress of a trenchless method. An installation technique should be selected that can accommodate removal of potential obstructions without having to remove or expose the leading edge of the encasement pipe.

7.6.2.3 Clogging Potential

According to clogging charts based on Atterberg's Limits testing of fat clay and till samples, our analysis indicates strong to medium clogging for fat clay layers and minimal clogging for glacial till. It is the contractor's duty to perform compatibility tests with various conditioning agents, dosing levels, and moisture contents to prevent clogging during field operations. Details on PL, LL, and moisture content are provided in the Geotechnical Data Report (GDR) and should be considered by the contractor before starting construction.

7.6.2.4 Void Development in Soil and Bedrock

The proposed siphon is anticipated to traverse through various layers, including clay, silt till, and bedrock, as identified in test holes TH24-01 to TH24-05. During the installation process, voids may develop both in the surrounding soil and within the bedrock, which is critical to understanding the potential impacts on stability and construction integrity.

In the soil, particularly within cohesive materials such as firm to very stiff clays, voids can form due to several factors, including excavation activities, soil settlement, and fluctuations in moisture content. These voids can lead to ground movement over time, potentially compromising the stability of the surrounding area. It is essential to monitor these conditions closely, as they can affect the performance of the siphon and the safety of the construction site.

Similarly, voids in bedrock may arise from natural geological processes, such as weathering and erosion, or from previous excavation activities. These voids can create challenges for the structural integrity of the siphon, as they may lead to unexpected ground movement or instability. Understanding the extent and nature of these voids is crucial for effective risk management during construction. As noted in Section 3.2, for the Northeast Interceptor Sewer Project. Karstic featuress (e.g. sinkholes, caves) could be encountered during tunneling in Winnipeg.

If significant voids are encountered in either the soil or bedrock, implementing circumference grouting outside the casing may be necessary to stabilize the ground and mitigate potential issues. This proactive approach helps ensure that the construction remains safe and effective.

Additionally, the contractor must ensure the proper installation of entry and exit seals at the break-in and break-out points of the trenchless crossing. This step is vital to prevent slurry loss prior to grouting, which can further safeguard against void formation and maintain the integrity of the installation process.

7.6.2.5 Bedrock Considerations

The proposed siphon is anticipated to be drilled through a bedrock layer with a lithology of lower Red River Formation; Dolomitic Mudstone, Brecciated. Understanding the geological characteristics of this formation is critical for the successful execution of the tunneling project. The Rock Quality Designation (RQD) of the bedrock ranges from poor to fair. This indicates the presence of fractures, which can lead to groundwater seepage. Additionally, cobbles and boulders may be encountered during tunneling operations. These conditions highlight the need for careful planning and mitigation strategies to address potential challenges related to bedrock stability and groundwater management.

Detailed bedrock test results, including unconfined compressive strength and CERCHAR Abrasivity tests, are available in the GDR, **Appendix I**.

The CERCHAR test is essential for evaluating the abrasiveness of rock materials, as this characteristic directly impacts the wear on cutting tools utilized in tunneling operations. Understanding the Abrasivity of the rock is critical for planning effective maintenance and tool replacement strategies, thereby ensuring the efficient operation of the MTBM. To mitigate the risks associated with rock abrasiveness, it is essential for the contractor to implement a comprehensive maintenance plan. This plan should include regular inspections and timely replacements of cutting tools to minimize downtime and ensure operational efficiency throughout the tunneling process.

7.6.2.6 Groundwater

As mentioned previously, Moderate to heavy groundwater seepage was observed in testholes TH24-01, TH24-02, TH24-04 and TH24-05 during drilling.

Groundwater readings were taken in testholes TH24-02 and TH24-04 upon the completion of drilling. Groundwater in testholes TH24-02 and TH24-04 was observed at depth of 11.4 m (Elev. 218.3 mASL) and 3.2 m (Elev. 226.1 mASL). Groundwater was measured and observed upon installation of the SP's in testholes TH24-01 and TH24-05. Groundwater level was monitored later from the SP's installed in testholes TH24-01 and TH24-05 within bedrock, details of groundwater readings are provided in Section 6 of the GDR. The installation of the siphon (top of siphon approx. 225.9 m ASL) is below the highest groundwater elevation recorded by the standpipe piezometer (SP) installed in TH24-05. During the construction of the jacking and receiving pit, the contractor should also be prepared to deal with groundwater originating from the till.

Groundwater will require careful management and control throughout the installation process. Groundwater can promote instability at the face of the tunnel boring machine and may result in higher ground deformations (settlement/heave) at ground surface unless adequate solutions are implemented. The contractor will have to develop a method to mitigate this risk especially if open-faced MTBM.

7.6.2.7 Pipe Alignment and Grade Control

Pipe alignment and grade control are critical during the initial stages of installation and require careful management to achieve adequate design inverts along the drive length. In difficult ground conditions where potential obstructions maybe present (i.e., abandoned pipes), encountering an obstruction may result in the reduction of alignment and grade control accuracy.

For tunneling machine, MTBM guidance system employs either an active laser guidance system, gyroscopic controls or advanced laser theodolite system to maintain the installation accuracy.

8. Design and Construction Considerations

8.1 General

Based on our current understanding of the proposed development and the results of our geotechnical investigation, the primary geotechnical concerns at the project site are:

- Based on the water levels recorded in standpipes SP24-01 and SP24-05, the water table will significantly influence the design and construction methods. As illustrated in **Figure 6.1**, the water measurement readings in SP24-01 and SP24-05 reflect the river's influence. The water elevation in the standpipes is higher than that of the river, and it decreases as you move towards the river which follows the general behaviour of the river and GW is influenced by the river. The approximate levels are as follows:
  - **June 2024:**
    - SP24-01 (Western Riverbank): 225.921 mASL
    - SP24-05 (Eastern Riverbank): 227.718 mASL
  - **January 2025:**
    - SP24-01 (Western Riverbank): 224.384 mASL
    - SP24-05 (Eastern Riverbank): 224.754 mASL
  - These variations in water table levels between June 2024 and January 2025 indicate seasonal fluctuations.
- Variable depths in bedrock depth.

8.2 Launch and Receiving Shafts

- The Contractor is responsible for the design of temporary support systems considered necessary for shafts in accordance with the Contract Documents.
- Two (2) vertical shafts are planned for construction as part of the proposed FGSV Siphon tunnel section. The launching and receiving shafts shall be located on the eastern and western side of the Red River (near TH24-03 and TH24-01), respectively. The shafts should be large enough to accommodate launching and retrieving of the MTBM, while providing space required for siphon construction as per Contract Drawings.
- Shafts will be used to launch and/or retrieve the MTBM and provide access and space for construction of the tunnel and permanent structures within the shafts. The shafts will be constructed in a combination of soil and bedrock.
- Due to proximity of buildings and utilities, use of temporary shoring will be required to support the excavation walls without impacting the adjacent structures.
- Ground movements are anticipated around the vertical shaft; therefore, the Contractor shall assess the potential adverse impacts and, where necessary, adopt suitable measures to prevent any damage to the utilities (underground and overhead) and buildings.
- The anticipated behavior of each type of soil/bedrock to be encountered is provided in **Table 7-1** Section 7.1 of this GBR.
- The baseline UCS for bedrock is provided in Section 7.4 of this GBR. The Contractor shall consider the UCS of bedrock for selecting equipment for bedrock excavation.
- For each shaft location, baseline elevations are presented in General Plan within the Contract Documents and Drawings.
- Temporary support and protection of the bedrock within the excavation should be provided as soon as possible after exposure to protect the bedrock from weathering, deterioration and spalling. Seepage at joints in the bedrock is expected.
- Temporary support systems are required to be designed for lateral earth pressure, lateral hydrostatic pressure, surcharge of equipment adjacent to the shaft, and should be capable of controlling ground movement in accordance with the Contract Documents. Shaft walls and base slab need to resist uplift forces

due to buoyancy, and adequate foundation details should be provided to prevent ground instability due to soil piping and basal heave. The following remarks regarding the receiving and jacking shaft can be used as baseline for the basal instability and buoyancy uplift from excess groundwater pressure beneath an impermeable stratum.

  - **Launch Shaft (East Riverbank):** Since the bottom of the excavation is found on the bedrock, excavation base stability is not a concern.
  - **Receiving Shaft (West Riverbank)**: As per Section 20.8.2.1 of the CFEM, base heave is deemed satisfactory if (FS) heave is greater than 1.5. The (FS) heave for the excavation of the proposed receiving shaft was calculated as 1.46 which is below a factor of safety of 1.5. The design of the temporary shoring system should be carried out by a professional engineer specialized in shoring design.

- All shoring designs should be in accordance with the 5th Edition of the Canadian Foundation Engineering Manual 2023 and must be reviewed by the design engineers. Surface surcharges from construction activities must be accounted for in the shoring design. If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding. The drilling contractor should account for potential for presence of obstruction in the till layer and at the bedrock surface when installing the shoring system. Cobbles and boulders are frequently encountered in the till layer above the bedrock.
- The construction of the shafts by “sealed” construction methods. The Contractor is required to submit their methods of designing and constructing a sealed shaft temporary support system to the Consultant for review with respect to meeting the performance requirements defined in the Contract Documents.
- The Contractor shall be prepared to collect and discharge potential seepage within the shafts and meet the discharge requirements indicated in the Contract Documents.
- As previously mentioned, the launch shaft is expected to be on top the bedrock at an approximate elevation of 216.40 mASL, while the receiving shaft is expected to be in clay at an elevation of 222.7 mASL. Therefore, there is the potential for boulders within the glacial till soil units and competent bedrock within the launch shaft excavation. It is anticipated that boulders less than 1 m^3 in size will be encountered. It will be necessary to use equipment that is robust enough to deal with these conditions during shaft excavation and shaft wall construction.
- The sealed shaft wall system selected by the Contractor shall be designed and constructed to allow for the Launch and receiving of the MTBM. This typically requires the incorporation of a “soft eye” reinforced with materials that can be cut by the MTBM along with a tunnel eye sealing system that prevents soil and groundwater ingress during MTBM breakout or breakthrough.
- The zone located outside of the shaft wall system at the break-in and break-out penetrations shall create a watertight zone where the MTBM can develop or dissipate earth pressure in the forward chamber of the MTBM and allow penetration through the shaft “soft eye”.

8.3 Tunnels

- The Contractor is to design the jacking pipes and construct the tunnel using a MTBM which can provide face support, installing and jacking pipes from the launching shaft immediately behind the MTBM.
- MTBM’s are to be used for the entire FGSV Siphon alignment in bedrock to install a large 2100 mm diameter RCP casing under the Red River in accordance with the Contract Documents.
- The anticipated face stability behavior of each soil unit to be encountered is provided in **Table 7-1** of this GBR.
- The cutter head should be designed to breakdown boulders and cobbles into fragments that are easily ingestible by the conveyance system (screw convey, slurry lines, etc.) or easily broken by a rock crusher.
- The MTBM is required to be utilised in conjunction with jacking pipe that provides full ground support over the entire excavated length of tunnel.
- Where the tunnel will be excavated in bedrock, the MTBM should be capable of boring through the following type of carbonated bedrock per our baseline interpretation:

  - Grade R5 (very strong) rock categorization according to ISRM Standard 1979
  - Medium CERCHAR-Abrasivity-Index according to ASTM D7625
- Watertight techniques are required to install the 2100 mm RCP or casing pipes in accordance with the Contract Documents, and this shall prevent significant groundwater inflow. Local dewatering or compressed air may be required to provide access to the face of the MTBM for maintenance, change of cutters, etc.
- The groundwater flow into the tunnel should be collected and discharged according to the requirements indicated in the Contract Documents.
- Contact grouting shall be used to completely fill the annulus between the ground and the lining to provide ground support and reduce ground settlement. Cementitious grouting is recommended to be done upon completion of each drive. To minimize surface settlement, all voids behind the lining must be completely filled with grout so that the tunnel lining is in direct contact with the ground.
- During microtunneling operations, bentonite or other suitable lubricating fluid should be used in the annular gap surrounding the pipe to minimize ground deformation and buildup of soil friction.
- To maintain face stability during excavation and avoid ground loss at the face it is essential that the chamber pressure is maintained within an acceptable range. Further, it is essential for the Contractor to ensure that the forward progress of the machine matches to the amount of excavation being removed from the chamber.
- MTBM selection should consider face intervention for tooling changes.

8.4 Impact on Existing Structures

Some degree of settlement, heave, and lateral movement will be an inevitable consequence of the construction of the shafts, tunnels, and there will also be some movement of adjacent structures and utilities. The Contractor shall undertake construction in a fashion which mitigates movements of utilities and structures within acceptable pre-defined limits, shown on Contract Drawings, to ensure there will be no adverse impacts or damage to the adjacent infrastructure.

During the tunneling process, minor ground loss may occur at the face of the MTBM, as well as some convergence of soil into the annular void surrounding the trailing pipes. These factors can lead to ground movements and settlements both longitudinally and transversely to the direction of tunneling.

To mitigate these potential impacts, it is essential that the Contractor implements appropriate risk management strategies throughout the operation. Continuous monitoring and adaptive measures will be crucial to ensure the stability of the surrounding ground and the integrity of the installation.With the selection of the MTBM as the trenchless method, it is anticipated that ground loss may occur at the tunnel face, along with some ground convergence into the annular space between the casing and the excavated tunnel walls. This can lead to ground movements and settlements both longitudinally and transversely to the tunneling direction. Therefore, it is important for the Contractor to implement effective risk mitigation strategies to address these potential issues and ensure the stability of the surrounding ground.

The contractor shall ensure that ground movements and settlements of adjacent utilities and buildings are maintained within acceptable limits. It is expected that the Contractor will adopt the following measures:

- Maintain the clearances indicated in the Contract Documents when tunnelling below or adjacent to utilities, buildings and the Red River.
- Minimise the magnitude of ground loss due to MTBM by:
  - Utilising an appropriate MTBM;
  - Utilizing appropriate trenchless methods for two tunnel sections required for the stub connections on east and west sides of the Red River;
  - Using experienced MTBM operators who will carefully control machine operating parameters for optimum results;
  - Limit the degree of radial overcut;

  - Fill the annulus with bentonite lubricant during microtunneling operations, and with cement grout immediately following completion of the tunnel drive;
- The Contractor should be highly experienced to avoid improper operation of the tunneling machine; and,
- Install and monitor the instrumentation shown on the Contract Documents and undertake investigation of MTBM operation and adopt suitable corrective measures in the event that instrumentation readings equal or exceed pre-defined alert levels.

8.4.1 Existing Structure and Potential Risks

In addition to the general impact's outlines above, specific existing structure such as embankments, multi-use path, and riverbanks present in the vicinity pose risks during tunneling operations.

8.4.1.1 Embankments

The stability of nearby embankments may be compromised due to ground movements associated with tunneling. The contractor must monitor these structures closely and implement stabilization measures if necessary to prevent erosion or collapse.

8.4.1.2 Multi-Use Paths

The construction activities may affect the integrity and usability of adjacent multi-use paths. The contractor should ensure that these paths remain safe and accessible through the construction process, providing detours or temporary closures as needed.

8.4.1.3 Riverbanks

The proximity of the Red River adds another layer of complexity. Ground movements could potentially lead to erosion or destabilization of the riverbanks, which may impact water flow and surrounding ecosystems. The contractor must take precautions to protect the riverbanks, including monitoring for signs and erosion and implementing protective measure as required.

8.5 Groundwater Management and Spoil Disposal

The Contractor shall be familiar with local spoil disposal regulations, and include the cost of all monitoring, testing, analyses, permits, and treatment necessary to meet the disposal guidelines as part of the Tender.

The Contractor's Environmental Construction Operations (ECO) Plan shall provide the methodology for managing impacted soils and groundwater, if encountered. The Contractor shall be responsible for managing and discharging groundwater in accordance with the applicable City of Winnipeg By-Laws and applicable provincial and federal regulatory requirements.

8.5.1 Groundwater Quality

8.5.1.1 Water Quality Sample Collection and Testing

Three water samples, including one field duplicate, were collected on February 6, 2025, from monitoring wells TH24-01 and TH24-05. Water samples were submitted to ALS Global (ALS) in Winnipeg, MB for analysis of benzene, toluene, ethylbenzene and xylene (BTEX), petroleum hydrocarbon fractions 1 and 2 (PHC F1-F2), styrene, polycyclic aromatic hydrocarbons, total metals, dissolved metals and select nutrient parameters. Tabulated analytical results are presented in **Tables 1** to **5.** Laboratory certificates of analysis are presented in **Appendix II**.

8.5.1.2 Applicable Guidelines

Guideline selection for groundwater analytical results was based on potential receiving environment governing authority. The City of Winnipeg Sewer By-Law No. 106/2018 (City of Winnipeg, 2022) lists contaminants of potential concern (COPCs) concentration limits for discharge to the wastewater system (Schedule B) and land drainage systems (Schedule D). Should the effluent be discharged directly to environment, quantitative limits set out in Tier III Manitoba Water Quality Standards, Objectives and Guidelines (Manitoba Water Stewardship, 2011) apply. Water quality guidelines are displayed in the tables.

8.5.1.3 Water Quality Results

8.5.1.3.1 BTEX, F1 and F2

Analytical results were below the detection limit for all BTEX, PHC F1-F2 and styrene parameters. Results are presented in Table 1 in **Appendix II**.

8.5.1.3.2 Polyaromatic Hydrocarbons

Analytical results were below applicable guidelines for all PAH parameters. Select PAH parameters were above the detection limit. Results are presented in Table 2 in **Appendix II**.

8.5.1.3.3 General Chemistry, Nitrogen and Phosphorus

Analytical results were below applicable guidelines for all nutrient and general chemistry parameters, except total phosphorous, which was above By-Law No. 106/2018 Schedule B and D limits. Unionized ammonia was not calculated. Ammonia will not be a trigger for concern except in water with high pH levels, which can be confirmed prior to construction. Results are presented in Table 3 in **Appendix II**.

8.5.1.3.4 Dissolved Metals

The Manitoba Water Quality Standards, Objectives and Guidelines are applied to dissolved metal analytical results. All results were below guidelines. Guidelines for select parameters are calculated based on water hardness which was not analysed. Results are presented in Table 4 in **Appendix II**.

8.5.1.3.5 Total Metals

Winnipeg's Sewer By-Law No. 106/2018 Schedule B and D limits apply to total metals. Select parameters were above limits, as summarized below.

Table 8-1: Summary of Total Metal Parameters

Total metal results exceeding Schedule B – Limits to Discharge into Wastewater System	Total metals result exceeding Schedule D – Limits to Discharge to Land Drainage System
Aluminum	Arsenic
Manganese	Chromium
Zinc	Copper
	Lead
	Manganese
	Nickel
	Zinc

Results are presented in **Table 5** in **Appendix II**.

8.5.1.4 Water Testing Quality Assurance

A quality assurance and quality control (QA/QC) program was implemented to minimize and quantify impacts introduced during sample collection, handling, shipping and analysis. As part of the QA/QC program, sampling protocols included minimizing sample handing, submitting field QA/QC samples, using dedicated sampling equipment, using sample-specific identification and labelling procedures and using chain of custody records.

One field duplicate sample was collected and submitted to the laboratory along with the original sample for analysis of the same parameters.

Laboratory QA/QC measures included analysis of duplicate and laboratory control samples. Details of the internal QA/QC procedures and methodologies employed by ALS are presented in the laboratory reports provided in **Appendix II**.

The field duplicate samples provide a means to evaluate the precision of the field quality control program. Reproducibility is quantified by calculating the relative percent difference (RPD) defined by the following equation:

$$Field\ Duplicate\ RPD\ (\%) = \frac{C1 - C2}{(C1 + C2)/_{2}} * 100$$

where: C1 = larger of the two observed values from the field duplicate analysis
C2 = smaller of the two observed values from the field duplicate analysis

Both sets of results must be greater than five times the laboratory reportable detection limit (RDL) to calculate a valid RPD.

All RPDs were below thresholds, and no QA/QC issues were identified. Parent and duplicate analytical results and calculated RPDs are presented in Table 6 in **Appendix II**.

8.5.1.5 Recommendations

Recommendations for this baseline water quality characterization include:

- The results of **Table 1** to **Table 5** in **Appendix II** could be used as a baseline for the groundwater quality.
- Water quality samples should be collected prior to any activities that will require groundwater withdrawal and disposal.
- Results from this baseline groundwater quality characterization should be reconfirmed to allow for proper planning and execution related to groundwater storage, conveyance and/or treatment prior to discharge.

9. Instrumentation Program

The potential impact of tunnel construction on adjacent structures should be monitored and instrumentation designed for the project location to monitor ground movements, settlement of any structures within the zone of influence, tunnel convergence, ground vibration, and level of noise. Details of instrumentation design, Review Level and Alert Level and amount of displacement/distortion that necessitate response for each level are provided in the Contract Documents (if any).

The potential impact of tunnel construction on the overlying ground, nearby buildings and other infrastructure will be monitored by the Contractor during construction.

9.1 Geotechnical Monitoring

Requirement for geotechnical monitoring are summarized as follows:

- Surface Monitoring Point (SMP), distributed along the tunnel route at points along the tunnel centerline on the east and west riverbanks.
- The Surface Monitoring Points will be supplemented by Settlement Monitoring Marker (SMM). These will primarily be in the multiuse path, crossing the tunnel route.
- Utility Monitoring Points (UMP) will monitor the settlement and will be installed near the following structures:
    - 1650 CONC LDS found south of the receiving shaft
    - 450 CSP found north of the launch shaft.
- Inclinometer (INC) are proposed at shaft locations to identify movement of shaft structures.
- Vibrating wire Piezometer (VWP) are proposed near shafts (mid-slope).

Instruments will be installed prior to the commencement of works to develop baseline values. The Tender documents specify review and alert levels for geotechnical monitors. These levels enable the Contractor to take necessary actions to prevent unacceptable movements, protecting the project and third-party structures, and providing data for third-party claims.

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Appendix I

Replacement of the FGSV Siphon Geotechnical Data Report

Replacement of the FGSV Siphon

Geotechnical Data Report
FINAL – Rev. 1

City of Winnipeg

607228226

April 2025

Delivering a better world

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The attached Report (the "Report") has been prepared by AECOM Canada ULC ("AECOM") for the benefit of the Client ("Client") in accordance with the agreement between AECOM and Client, including the scope of work detailed therein (the "Agreement").

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AECOM: 2015-04-13

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Appendices

1. Introduction

1.1 General

AECOM Canada ULC was retained by the City of Winnipeg Water and Waste Department (the City) to provide geotechnical engineering services to support the design and construction of the proposed Fort Garry- St Vital (FGSV) Siphon that crosses the Red River. The project site is located at the Fort Garry Bridge, Winnipeg, MB. The Fort Garry Bridge is a paired bridge system, with the north bridge serving westbound traffic and the south bridge serving eastbound traffic. AECOM understand that installation of the proposed FGSV Siphon below the Red River will be completed using either micro-tunneling or horizontal directional drilling (HDD), from the western siphon outlet chamber to the eastern siphon inlet chamber.

This Geotechnical Data Report (GDR) presents the results of a detailed geotechnical investigation conducted by AECOM along the proposed FGSV Siphon alignment. The detailed geotechnical investigation was conducted in general accordance with the American Society of Civil Engineers (ASCE) Manual of Practice 154 Geotechnical Baseline Reports: Suggested Guidelines.

This report also provides a summary of previous geotechnical investigation program undertaken near the site. The results and factual outcomes of this study are included within Section 2 of this report.

This GDR should be read in conjunction with the Geotechnical Baseline Report (GBR). The GDR is subject to AECOM's Statement of Qualification and Limitations and General Statement regarding the Normal Variability of the Subsurface Conditions.

1.2 Aims and Objectives

The main objectives of the geotechnical investigation were to determine the subsurface soil/bedrock/groundwater conditions and engineering properties of the soil/bedrock encountered at the test hole locations drilled along the FGSV alignment. The primary focus of this report is to present and document factual findings from AECOM and other relevant geotechnical investigations and laboratory testing programs. The results of AECOM's laboratory testing program and test hole logs are included within this report.

The analyses and results presented in this report are based on the data obtained from the test holes drilled at distinct locations along the FGSV alignment. This report does not reflect any variations which may occur between the test hole locations. In the performance of subsurface explorations, specific information is obtained at specific locations at specific times. However, it is well known that variations in soil, bedrock, and groundwater conditions exist at most sites between test hole locations. The nature and extent of the variations may not become evident until the course of construction. If variations are then evident, it will be necessary to re-evaluate the findings and results presented in this report after performing on-site observations during the construction period and noting the characteristics of any variations.

1.3 Project Details

The FGSV Siphon replacement project involves the replacement of the failed 700 mm wastewater siphons crossing the Red River between the Abinoji Mikanah east bound and west bound bridges.

The new FGSV siphon replacement will be installed using a trenchless method, which will consist of either micro tunnel boring machine (MTBM) technology or horizontally directionally drilled (HDD) method. Both methods involve tunneling underneath the river, beginning at the entry pit (near testhole TH24-05) and exiting at the exit pit (near testhole TH24-01). The following trenchless installation approach ensures minimal disruption to surface activities and infrastructure while efficiently replacing critical underground infrastructure:

1. MTBM Technology: A large 2100 mm diameter reinforce concrete pipe (RCP) casing installed beneath the river in bedrock, with two 900 mm DR11 HDPE pulled through after the casing install; or
2. Horizontally Directionally Drilling (HDD): Twin 900 mm DR9 HDPE pipes will be installed using HDD beneath the river in bedrock.

In addition to the trenchless river crossing, new 1350 mm RCP will be installed using trenchless pipe jacking methods to connect the siphon crossing at two locations:

- Approx. 60 m from the discharge manhole to the upstream siphon chamber on the west side of the Red River.
- Approx. 60 m from the downstream siphon chamber to the existing St. Vital Trunk.
    a) Photographs of the project site taken at the time of the field drilling program are provided in **Appendix 1**.

1.4 Scope of Work

The scope of work for the detailed geotechnical investigation along the FGSV alignment is summarized below:

1. Review of geological survey maps and relevant background information.
2. Obtain and review geotechnical reports provided to AECOM with respect to the subject site. AECOM will also review geotechnical reports available in AECOM's library to collect information on the soil and bedrock within and near to the subject site.
3. Prepare a GDR that documents the findings from AECOM's 2024 investigation and from previous geotechnical investigations and laboratory testing.

2. Background Information

2.1 Review of Background Reports

A review of available geotechnical information pertinent to the project was conducted including the geotechnical report prepared by AECOM Canada Ltd. (2021). The main objective of the review was to obtain and present information specific to the subsurface conditions, groundwater conditions and riverbank stability with respect to the FGSV alignment. The available memorandum was reviewed to prepare a GDR that presents the factual information collected from the site investigation and laboratory testing. The following information was provided to the project team by the City:

- AECOM Canada Ltd. (2021). City of Winnipeg High Risk River Crossing – Phase 3 – Geotechnical Condition Assessment.
- AECOM Canada Ltd. (2018). City of Winnipeg Geotechnical Assessment Ft. Garry-St. Vital Feeder Main

Appendix 2 shows the locations of test holes from the past and current investigations relevant to the site. This information was reviewed to improve the understanding of site conditions and riverbank stability during the construction of the existing Fort Garry-St. Vital Interceptor Siphon, located approximately 55 to 65 m north of the proposed siphon location.

In summary, the review indicated the following:

- The riverbank soil consists of lacustrine and alluvial layers overlying glacial till and limestone bedrock.
- Stabilization measures will likely be required for the west riverbank if disturbed during construction.
- Constructability challenges (sloughing, seepage etc.) are anticipated, dewatering and temporary shoring will be required.
- Bedrock contains zones of large fractures and weak rock.
- Ground stabilization (1989/90) was completed on the west bank adjacent to the existing bridge location.

2.2 Background Information from AECOM (2021)

The geotechnical condition assessment for Site 4, the existing Fort Garry Bridge Siphon Crossings, involved reviewing available background information and conducting a visual field inspection within a 30 m zone around the crossing. The assessment aimed to evaluate potential risks of slope instability and erosion affecting the buried sewer and water systems.

As noted in the Technical Memorandum (AECOM, 2021), the findings from the review and inspection were used to assign Slope Condition Grade (SCG) and Erosion Condition Grade (ECG), helping to determine the need for further geotechnical investigation or slope stability analysis. The results are detailed in the Technical Memorandum, which includes the assigned condition grades and any additional geotechnical findings. The Technical Memorandum is found in **Appendix 6**.

Available Background Information Review

The available background information covers geotechnical investigations conducted at six different sites throughout the city of Winnipeg. This review focuses on Site 4, located at the Abinoji Mikanah Bridge crossing on the Red River in south Winnipeg. Site 4 features two bridge structures and pedestrian crossings. The Fort Garry-St. Vital interceptor siphons, with diameters of 700 mm and 800 mm, are embedded in alluvial sediments on the banks and surface laid across the bottom of the river. Geotechnical investigations from 1975-76 and 2013 indicated that the slope of the eastern riverbank was unstable under rapid drawdown conditions, posing a risk to the 800 mm siphon. Recommendations for slope stabilization, including placing stone riprap and regrading, to protect the existing siphon pipe, were implemented in 2014.

Site Reconnaissance

On November 17 and 18, 2020, AECOM conducted a visual inspection for the riverbanks at Site 4, focusing on both the west and east riverbanks.

West Bank:

- Observed minor erosion scarps and a scarp near the crest are likely from shallow failures. No deep-seated failures were noted. The bank is classified as altered due to localized ripraps around the toe. The riprap was large and moving, with some erosion and gullying around bridge abutments.
- The slope profile ranged from 2H:1V to 3H:1V, with erosion scarps 100-150 mm high in unarmored areas. No evidence of deep-seated instabilities or animal burrows was found.

East Bank:

- Minor erosion was observed above the riprap, which was placed in 2013. The bank is also classified as altered. The slope profile ranged from 3H:1V to 4H:1V. Some riprap was missing around bridge piers, exposing alluvial soils.
- Erosion scarps 100 mm high were noted in unarmored areas. No deep-seated slope instabilities or animal burrows were observed, though animal burrows were noted east of the sidewalk.

Overall, both banks exhibited localized erosion and required further stabilization, but no significant instability or damage to structures was detected. **Table 2-1** provides a summary of the SCG and ECG rating selected for each bank at this site.

Table 2-1: Summary of SCG and ECG Values (Site 4 – AECOM 2021)

Riverbank	SCG[1]	ECG[2]	Comments
West	3	2	Evidence of slope instabilities and erosion indicated need for further analysis. Slope stability analysis completed at this site and results presented below.
East	1	2	No defects observed with slope condition. Minor erosion observed, short-term potential for further deterioration of asses due to slope instability and erosion is low.

1. SCG = Slope Condition Grade.
2. ECG = Erosion Condition Grade.

Geotechnical Investigation

Based on the results of the background information review and the visual field inspection, it was deemed that Site 4 did not require geotechnical investigation, laboratory testing and instrumentation installation/monitoring.

Slope Stability

To develop the slope stability model for the west riverbank at Site 4, subsurface data from test holes 1003, 1004, and 401: Klohn Leonoff Consultants Ltd. (April 12, 1976) were utilized.

Shear strength values were assigned to the alluvial and glacio-lacustrine clay layers, with bedrock treated as impenetrable and riprap not included in the analysis due to limited data. The parameters used for the stability analysis are shown in **Table 2-2**.

Table 2-2: Geotechnical Parameters Used in Slope Stability Modelling (Site 4 – AECOM 2021)

Soil Description	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (°)
Alluvial Clay	18	18	5
Glacio-Lacustrine Clay	18	14	5
Glacial Till	21	30	10.0

Slope stability analyses were completed for the west bank and the FS values results from the analyses are presented in **Table 2-3**.

Table 2-3: Riverbank Slope Stability Results Along Pipe Alignment (Site 4 – AECOM 2021)

File Output Reference	Slope Stability Case	Factor of Safety (FS)
West		West
H-01	Long Term – Normal Winter Water Level (NWWL)	1.39
H-02	Long Term – Normal Summer Water Level (NSWL)	1.46
H-03	Short Term – Rapid Draw Down (RDD)	1.30

Based on the results of the preliminary slope stability assessment for Site 4, the following general conclusions and recommendations are summarized:

- For long-term conditions, the FS values indicate a risk of failure affecting the HDPE interceptor sewers, though the risk is low. The short-term FS value meets the industry standard of 1.30.
- Long-term FS values are below the standard FS of 1.5, but immediate slope failure is unlikely. Regular monitoring of slope stability due to erosion is recommended.
- Slope improvements should be evaluated on a cost/benefit basis. Short-term actions may include visual inspections or instrumentation monitoring (e.g., slope inclinometer) for ground movements, if needed, slope regarding and expanded riprap placement around the crossing.

3. Geotechnical Investigation

3.1 Drilling and Sampling Program

AECOM obtained underground service clearances from public utility companies (Click Before You Dig Manitoba). A utility locator identified and marked the private utilities on May 20, 2024. The subsurface drilling and sampling program was conducted from June 3 to June 7 and August 9, 2024. Drilling services were provided by Paddock Drilling under the supervision of AECOM geotechnical field personnel. The proposed testholes are shown on the attached location plan provided in **Appendix 2**. Five (5) testholes were drilled on the project sites using a track mounted and barge drill rig which was equipped with 125 mm solid stem augers and HQ coring. Testholes TH24-01 and TH24-05 were cored into the bedrock at depths of 26.14 m and 24.69 m within the site area, while TH24-03 was cored into the bedrock at a depth of 35 m, respectively. Testholes TH24-02, and TH24-04 were drilled to auger refusal within the site area, at depths of 12.95 m and 13.11 m. Sloughing was observed in testholes TH24-01, TH24-02 and TH24-04, at a depth between 9.14 m and 16.46 m.

Soil samples were obtained directly from the auger flights at depth intervals ranging from 0.3 to 1.5 m. SPT were conducted in testhole TH24-02 to assess the relative density of cohesionless soils. The soil samples were visually classified in the field and returned to our soil laboratory for additional examination and testing. Cohesive soil samples were tested using a pocket torvane and penetrometer to estimate the undrained shear strength and the compressive soil strength.

Upon completion of drilling, the testholes were examined for evidence of sloughing and groundwater seepage, sealed with bentonite at the bottom, and the excess auger cuttings were left on site. The detailed testhole records are provided in **Appendix 3**, which include a summary sheet outlining the symbols and terms of the testhole record.

3.2 Groundwater Levels Monitoring

During the geotechnical field investigation, two (2) standpipe piezometers (SP) consisting of 50 mm in diameter and 305 mm in length screening Casagrande tip were installed. The installation details of the standpipe piezometers are shown on the testhole logs in **Appendix 3** and summarize in **Table 3-1.**

Table 3-1 :Standpipe Piezometer Installed for GWL Reading

Testhole No.	SP depth (m)	Tip Elevation (m ASL)	USCS Soil Type
TH24-01 (SP1)	25.2 m	208.58	Bedrock
TH24-05 (SP5)	24.7 m	207.21	Bedrock

4. Laboratory Testing

A laboratory testing program was performed on soil samples obtained during the drilling program to determine the relevant engineering properties of the subsurface materials. The laboratory tests consisted of geotechnical testing on disturbed and bulk samples. The geotechnical tests were conducted at Geomechanica's Materials Testing Laboratory in Oakville, Ontario, as well as at the Materials Testing Laboratories of AECOM and Eng-Tech in Winnipeg, Manitoba. In addition, pocket torvane readings were taken on auger grab samples. The results of the laboratory testing are shown on the testhole records in **Appendix 2** and on the laboratory test reports in **Appendix 3**.

4.1 Geotechnical Testing

Geotechnical laboratory testing was performed on selected soil samples to evaluate the physical characteristics, evaluate the engineering properties and aid with further characterization of the subsurface. The geotechnical laboratory testing program included diagnostic testing included moisture contents on all collected soil samples, as well as particle size analysis, Atterberg limits tests, unconfined compressive strength on clay, unconfined compressive strength of intact rock core, and abrasiveness of rock on some samples. A summary of the geotechnical testing that was completed in **Table 4-1.** The results of the laboratory testing are shown on the testhole records in **Appendix 3** and within the laboratory test reports in **Appendix 4**.

Table 4-1: Summary of Laboratory Testing

Laboratory Test	Number of Tests	Testing Standard
Moisture Content	60	ASTM D2216
Particle Size Analysis (Hydrometer Analysis)	15	ASTM D422
Atterberg Limits	15	ASTM D4318
Unconfined Compressive Strength (Clay)	10	ASTM D2850
Unconfined Compressive Strength of Intact Rock Core	5	ASTM D2938
Abrasiveness of Rock Using the CERCHAR Abrasiveness Index Method	5	ASTM D7625

5. Subsurface Conditions

Subsurface conditions observed during testhole drilling and sampling were visually documented by AECOM geotechnical personnel in accordance with the Unified Soil Classification System (USCS).

The conditions of the site have been based on the investigation results obtained during the field and laboratory investigation programs. The pertinent results from these investigations are outlined below.

5.1 Subsurface Profile

The soil stratigraphy on the project site generally consists of topsoil, clay fill overlying a clay deposit, which is underlain by sand till and bedrock. Additionally, alluvial deposits are observed at the riverbank and along the river bottom. A description of the soil stratigraphy is provided below. The detailed testhole records are provided in **Appendix 3**, which include a summary sheet outlining the symbols and terms of the testhole record.

5.1.1 Topsoil

Topsoil was encountered at the ground surface in testholes TH24-01, TH24-02, TH24-04, and TH24-05. The thickness of the topsoil was approximately 0.30 m and is observed to be black, moist, with organic content, with traces of sand, gravel, and silt. The moisture content of the topsoil ranged from 31.4% to 35.6%.

5.1.2 Fill – Clay (CL)

Black fat clay (CL) fill material was encountered in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from approximately 0.7 m to 1.9 m. The clay (CL) fill layer was generally observed to be moist, high plasticity, black in color, firm to stiff and have traces of sand, gravel, and silt. The moisture content of the clay fill (CH) fill ranged from 32.8% to 35.6%.

5.1.3 Clay (CH)

Grey fat clay (CH) was encountered below the clay fill materials in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from 10.10 to 15.75 m. It is observed to be moist, firm, and high plasticity with silt inclusions. The clay shear strength varies from firm to soft and decreases with depths. The moisture content of the fat clay (CH) ranged from 13.6% to 51.3%.

5.1.4 Silt (ML) Till

Tan silt (ML) till was encountered below the clay fill material in TH24-01, TH24-02, TH24-04, and TH24-05, with a thickness ranging from 0.71 m to 1.95 m. It is observed to be moist, loose, and of low plasticity with trace of sand, clay and gravel. The silt shear strength was soft. The moisture content of the silt (ML) till ranged from 11.4% to 18.5%.

5.1.5 Bedrock

Bedrock (BR) was encountered below the silt (ML) in the cored testhole TH24-01, TH24-03 and TH24-05. Brecciated Dolomitic Mudstone was the type of rock observed in the coring, a Lower Fort Garry Member of the Red River Formation. The Brecciated Dolomitic Mudstone was observed at the depth of 216.38 and 217.20 m ASL to beyond 207.20 m ASL and 182.53 m ASL. During coring, it was observed that there was no water return. The lack of water return typically indicates the presence of large fractures within the bedrock. The dolomitic limestone was white greyish to dark grey and was nodular bedded. The quality and strength of the bedrock will be discussed further in Section 7.4. Section 7.4.1 describes the total core recovery (TCR), Section 7.4.2 describes the solid core recovery (SCR), Section 7.4.3 describes the rock quality designation (RQD), and Section 7.4.3 describes the bedrock classification results.

5.1.6 Clay Deposition

5.1.6.1 Alluvial Deposits

Based on the meandering of the river, we anticipate that the river overburden will primarily consist of alluvial deposits, mainly made up of clay, silt, sand, and organic materials. The meandering of the river creates an alluvial deposit on the west side and lacustrine deposit on the east riverbank. The properties and classifications of these materials may differ. The extent of these alluvial deposits is not well-defined, because the drilling operations focused solely on reaching the targeted bedrock depth and did not include sampling or testing of the overburden.

5.1.6.2 Lacustrine Deposits

Lacustrine deposits, which form in glacial lakes, were found in the project area. The Glacio-Lacustrine clay in the area varies in thickness. The clay layer tends to be thinner near the river channel and increases in thickness as the distance from the river channel increases. The clay is thinner in the eastern riverbank compared to those located along the western riverbank. Additionally, the meandering of the river creates an alluvial deposit on the west side and lacustrine deposit on the east riverbank.

6. Groundwater and Sloughing Conditions

Groundwater seepage or soil sloughing conditions were observed in most testholes upon completion of drilling. Details of the location and nature of the sloughing, seepage, and groundwater encountered are provided on the testhole logs in **Appendix 3** and presented in **Table 6-1**.

Table 6-1: Observed Groundwater Seepage and Sloughing Conditions

Testhole No.	Groundwater Seepage	Depth of Groundwater Seepage (m)	Groundwater Depth Upon Completion of Drilling (m)	Depth of Soil Sloughing
TH24-01	Moderate	9.0	7.9	14.3 m & 16.5 m
TH24-02	Heavy	10.4	11.4	11.0 m & 11.4 m
TH24-04	Heavy	9.1	3.2	9.1 m & 12.2 m
TH24-05	Moderate	6.1	5.1	None

6.1 Standpipe Piezometer Monitoring Results

Groundwater readings were taken upon completion of the testhole drilling and utilizing the standpipes installed in TH24-01 (SP24-01) and TH24-05 (SP24-05) by AECOM. The readings recorded are summarized in **Table 6-2**.

Table 6-2: Groundwater Readings

Standpipe	Groundwater Elevation (m ASL)								
	Stratum/Tip m ASL	Jun. 4/24	Jun. 6/24	Jun. 10/24	Jun. 11/24	Jun. 17/24	Jun. 24/24	Jan. 30/25	Mar. 12/25
SP24-01	Bedrock/207.70	225.89	-	226.06	-	225.94	225.78	224.38	223.87
SP24-05	Bedrock/207.20	-	226.78	-	226.90	226.69	226.50	224.75	225.92

Normal River Level (Summer) = 223.98 m ASL

A graphical summary of these results is provided in **Figure 6-1.**

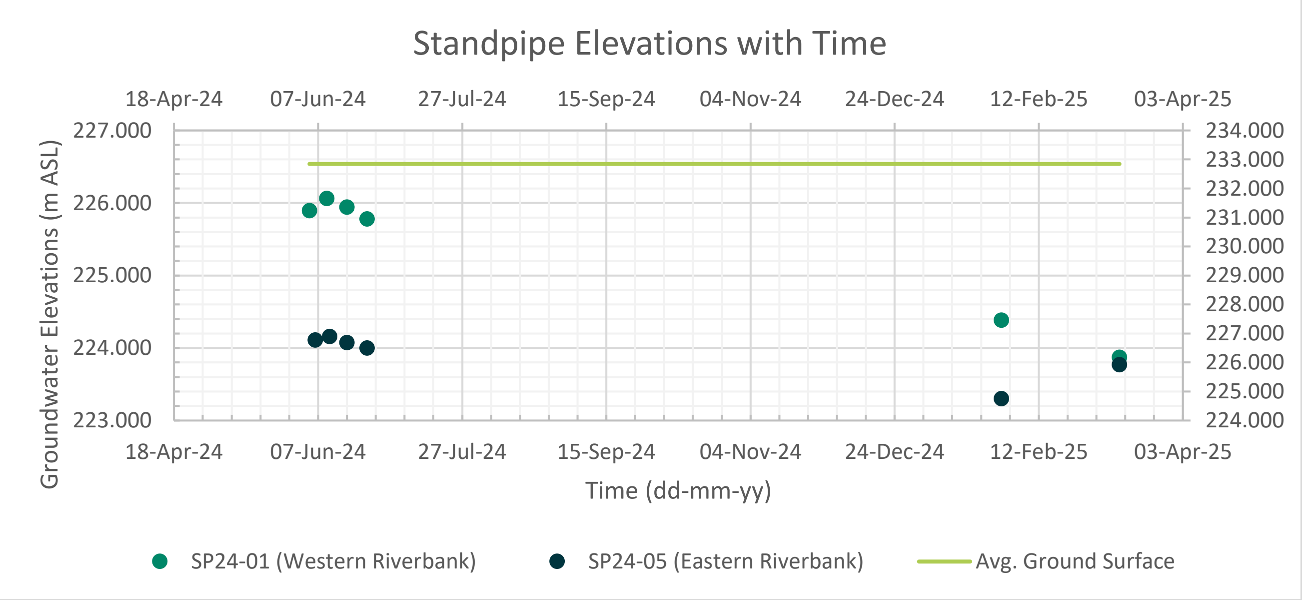

Figure 6-1: Graph of Groundwater Elevations Versus Time

Only short-term seepage and sloughing conditions were observed in the testholes. Groundwater levels will normally fluctuate during the year and will be dependent on precipitation, surface drainage, and regional groundwater regimes. Groundwater seepage and soil sloughing should be expected from the silt (ML) till layer and expected in entry and exit pit excavations during construction.

7. Laboratory Testing Results

7.1 General

Samples retrieved from the testholes were selected for geotechnical laboratory testing to characterize material types and determine their engineering properties.

7.2 Overburden Soils

Table 7-1: Particle Size Analysis

Testhole No.	Sample Depth (m)	Group Name	Particle Size			
			Gravel 75 to 4.75 mm	Sand <4.75 to 0.075 mm	Silt <0.075 to 0.002 mm	Clay <0.002 mm
TH24-01	0.61 – 0.76	CH	0.0%	1.6%	28.9%	69.5%
TH24-01	4.42 – 4.57	CH	0.0%	1.3%	38.9%	59.8%
TH24-01	10.52 – 10.67	CH	0.2%	2.2%	35.2%	62.5%
TH24-01	16.61 – 16.76	CL-ML	10.4%	33.5%	41.7%	14.4%
TH24-02	5.94 – 6.10	CH	0.0%	1.4%	50.4%	48.1%
TH24-02	10.52 – 10.67	CH	0.0%	0.2%	32.1%	67.8%
TH24-02	12.04 – 12.19	CL	4.6%	33.6%	43.6%	18.1%
TH24-04	5.94 – 6.10	CH	0.0%	1.7%	47.6%	50.6%
TH24-04	8.99 – 9.14	CH	0.0%	1.1%	45.3%	53.5%
TH24-04	12.04 – 12.19	CH	3.4%	5.9%	32.0%	58.7%
TH24-04	12.95 – 13.11	CL	2.4%	26.9%	49.1%	21.5%
TH24-05	0.76 – 0.91	CH	0.0%	0.9%	44.6%	54.6%
TH24-05	4.42 – 4.57	CH	0.0%	0.1%	47.8%	52.1%
TH24-05	10.52 – 10.67	CH	0.2%	1.6%	35.0%	63.2%
TH24-05	13.58 – 13.72	CL	8.0%	36.8%	38.9%	16.2%

Table 7-2: Atterberg Limits Test Data

Testhole No.	Sample Depth (m)	USCS	Liquid Limit	Plastic Limit	Plasticity Index
TH24-01	0.61 – 0.76	CH	84	22	62
TH24-01	4.42 – 4.57	CH	90	26	64
TH24-01	10.52 – 10.67	CH	85	24	61
TH24-01	16.61 – 16.76	CL-ML	15	11	58
TH24-02	5.94 – 6.10	CH	80	24	56
TH24-02	10.52 – 10.67	CH	92	24	68
TH24-02	12.04 – 12.19	CL	21	12	9
TH24-04	5.94 – 6.10	CH	86	23	63
TH24-04	8.99 – 9.14	CH	81	22	59
TH24-04	12.04 – 12.19	CH	67	18	49
TH24-04	12.95 – 13.11	CL	27	12	15
TH24-05	0.76 – 0.91	CH	91	27	64
TH24-05	4.42 – 4.57	CH	96	23	73
TH24-05	10.52 – 10.67	CH	74	21	53
TH24-05	13.58 – 13.72	CL	18	10	8

Table 7-3: Unconfined Compressive Strength Test (Soil)

Testhole No.	Sample Depth (m)	Soil Type	Moisture Content (%)	Undrained Shear Strength (kPa)	Unconfined Compressive Strength (kPa)
TH24-01	3.05 – 3.66	CH	13.6	73.09	146.18
TH24-01	6.10 – 6.71	CH	15.0	29.06	58.12
TH24-01	12.19 – 12.80	CH	47.3	49.23	98.45
TH24-02	3.05 – 3.66	CH	33.4	74.65	149.31
TH24-02	9.14 – 9.75	CH	32.7	68.37	136.74
TH24-04	3.05 – 3.66	CH	14.6	48.97	97.93
TH24-04	9.14 – 9.75	CH	33.1	50.09	100.19
TH24-05	1.52 – 2.13	CH	14.2	95.63	191.25
TH24-05	7.62 – 8.23	CH	32.1	52.67	105.34
TH24-05	10.67 – 11.28	CH	16.1	30.87	61.74

7.3 Bedrock

Table 7-4: Unconfined Compressive Strength of Intact Rock Core Specimens Results

Testhole No.	Sample Depth (m)	Sample Elevation (m ASL)	Maximum Load (kN)	Compressive Strength (MPa)
TH24-01	18.3 – 18.5	215.48 – 215.28	243.3	78.0
TH24-03	16.29 – 16.49	207.69 – 207.49	291.8	93.0
TH24-03	17.46 – 17.71	206.52 – 206.2	734.5	235.0
TH24-03	29.97 – 30.19	194.01 – 193.79	273.4	87.7
TH24-03	31.43 – 31.65	192.55 – 192.33	157.7	50.6
TH24-03	32.28 – 32.76	191.70 – 191.22	110.0	35.3
TH24-05	23.75 – 24.2	208.16 – 207.71	398.5	128.0

Table 7-5: CERCHAR Abrasive Test Results

Testhole No.	Sample Elevation (m ASL)	Test 1 Mean (mm)	Test 2 Mean (mm)	Test 3 Mean (mm)	Test 4 Mean (mm)	Test 5 Mean (mm)	Mean Wear (mm)	CAI	Lithology	ASTM Classification
TH24-01, C23	208.35 – 207.35	0.127	0.068	0.105	0.176	0.165	0.128	1.281	Lower Red River Formation: Dolomitic Mudstone, Brecciated	Medium
TH24-03, C09	207.85 – 207.69	0.138	0.165	0.179	0.186	0.179	0.169	1.694	Lower Red River Formation: dolomitic mudstone, brecciated	Medium
TH24-03, C10	206.71 – 206.52	0.157	0.152	0.140	0.151	0.159	0.152	1.517		Medium
TH24-03, C20	194.87 – 194.69	0.117	0.114	0.050	0.040	0.073	0.079	0.789		Low
TH24-03, C21	192.85 – 192.66	0.059	0.055	0.029	0.034	0.034	0.042	0.423		Very Low
TH24-03, C22	191.14 – 190.99	0.046	0.051	0.048	0.080	0.029	0.051	0.509		Very Low
TH24-05, C23	208.48 – 208.30	0.154	0.164	0.167	0.164	0.190	0.168	1.677	Lower Red River Formation: Dolomitic mudstone, brecciated	Medium

7.4 Bedrock Classification

The rock strength can be categorized with the unconfined compressive strength of the rock based on International Society of Rock Mechanics (ISRM) Standard (1979) as shown in **Table 7-6**. AECOM prepared two (5) rock specimens for the unconfined compressive strength of intact rock tests to be processed for testing.

Table 7-6: Rock Strength Categorization

Grade	Term	Unconfined Compressive Strength (MPa)
R6	Extremely Strong	>250
R5	Very Strong	100 – 250
R4	Strong	50 – 100
R3	Medium Strong	25 – 50
R2	Weak	5 – 25
R1	Very Weak	1 – 5
R0	Extremely Weak	0.25 – 1

The testing results for the TH24-01 (C18) sample showed an unconfined compressive strength of 78 MPa. For the TH24-03 (C20, C21, and C22) samples, the unconfined compressive strengths were 87.7 MPa, 50.6 MPa, and 35.3 MPa, respectively. The TH24-05 (C23) sample exhibited an unconfined compressive strength of 128 MPa. Based on these results, AECOM concludes that the rock strength ranges from medium strong (R3) to very strong (R5).

7.4.1 Total Core Recover (TCR)

Total core recovery (TCR) is the testhole core recovery percentage. TCR is expressed as follows:

$$TCR\ (\%) = \frac{sum\ of\ recovered\ core\ length}{total\ core\ length}\ x\ 100$$

The TCR was calculated for each bedrock core run advanced within the testholes. A summary of the TCR values is provided in **Table 7-8**.

7.4.2 Solid Core Recover (SCR)

Solid core recovery (SCR) is the testhole core recovery percentage of solid cylindrical rock. SCR is expressed as follows:

$$SCR\ (\%) = \frac{sum\ of\ recovered\ solid\ cylindrical\ core\ lengths}{total\ core\ length}\ x\ 100$$

The SCR was calculated for each bedrock core run advanced within the testhole. A summary of the SCR values is provided in **Table 7-8**.

7.4.3 Rock Quality Designation (RQD)

RQD is based on the ISRM classification System. The RQD is an indirect measure of the number of fractures and the amount of jointing in the rock mass. The RQD is expressed as a percentage of the ratio of summed core lengths (greater than 10 cm) to the total length cored. The RQD index is used to provide a classification of the rock quality shown in **Table 7-7**.

Table 7-7: Rock Classification Ranges

RQD (%)	Rock Quality Designation
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Rock quality designation (RQD) is expressed as follows:

$$RQD\ (\%) = \frac{sum\ of\ recovered\ core\ lengths\ greather\ than\ 10\ cm}{total\ core\ length}\ x\ 100$$

The RQD was calculated for each core run advanced within TH24-01, TH24-03 and TH24-05. A summary of the RQD values is provided below in **Table 7-8**.

7.4.4 Bedrock Classification Results

Based on the rock classification and laboratory test results (as shown in **Table 7-4**) the encountered bedrock classification ranges from very poor to excellent quality, with a range of intact rock strength from extremely weak (R0) to strong (R4).

Table 7-8: TCR, SCR, and RQD Results

Testhole ID	Sample Number	Core Run No.	Core Run Depth (m bgs)	Elevation (m asl)	TCR (%)	SCR (%)	RQD (%)
TH24-01	C18	1	17.37 - 18.52	216.41 - 215.26	94	78	67
	C19	2	18.52 - 20.04	215.26 - 213.74	93	71	57
	C20	3	20.04 - 21.56	213.74 - 212.22	79	22	20
	C21	4	21.56 - 23.09	212.22 - 210.69	97	79	78
	C22	5	23.09 - 24.61	210.69 - 209.17	84	54	45
	C23	6	24.61 - 26.14	209.17 - 207.64	81	76	68
TH24-03	C1	1	8.23 - 8.69	209.35 - 208.89	61	28	0
	C2	2	8.69 - 9.14	208.89 - 208.44	95	97	53
	C3	3	9.14 - 10.67	208.44 - 206.91	96	81	47
	C4	4	10.67 - 12.19	206.91 - 205.39	90	71	41
	C5	5	12.19 - 13.72	205.39 - 203.86	98	96	81
	C6	6	13.72 - 14.27	203.86 - 203.31	91	68	68
	C7	7	14.27 - 15.24	203.31 - 202.34	87	80	56
	C8	8	15.24 - 15.85	202.34 - 201.73	96	82	72
	C9	9	15.85 - 16.76	201.73 - 200.82	94	88	86
	C10	10	16.76 - 18.29	200.82 - 199.29	96	75	57
	C11	11	18.29 - 19.81	199.29 - 197.77	98	86	64
	C12	12	19.81 - 20.93	197.77 - 196.65	91	88	84
	C13	13	20.93 - 21.34	196.65 - 196.24	93	65	39
	C14	14	21.34 - 22.86	196.24 - 194.72	88	73	60
	C15	15	22.86 - 23.93	194.72 - 193.65	87	70	70
	C16	16	23.93 - 25.15	193.65 - 192.43	92	66	62
	C17	17	25.15 - 25.91	192.43 - 191.67	94	90	90
	C18	18	25.91 - 27.43	191.67 - 190.15	98	86	84
	C19	19	27.43 - 28.96	190.15 - 188.62	98	81	73
	C20	20	28.96 - 30.48	188.62 - 187.10	97	70	59
	C21	21	30.48 - 32.00	187.10 - 185.58	98	90	83
	C22	22	32.00 - 33.53	185.58 - 184.05	99	98	89
	C23	23	33.53 - 35.05	184.05 - 182.53	97	96	94
TH24-05	C17	1	14.73 - 15.49	219.05 - 218.29	69	0	0
	C18	2	15.49 - 17.02	218.29 - 216.76	78	30	25
	C19	3	17.02 - 18.54	216.76 - 215.24	81	32	29
	C20	4	18.54 - 20.07	215.24 - 213.71	94	85	58
	C21	5	20.07 - 21.59	213.71 - 212.19	92	70	62
	C22	6	21.59 - 23.11	212.19 - 210.67	96	88	87
	C23	7	23.11 - 24.69	210.67 - 209.09	89	85	80

TH24-01: all six (6) core runs exhibited good recovery runs, with varying rock classification; C18, C19, C21, and C23 exhibited a fair rock classification. While C20 and C22 exhibited a very poor and poor rock classification.

TH24-03: all twenty-three (23) core runs exhibited good recovery runs, with varied rock quality designations; C1 exhibited a poor rock quality designation. C2, C6, C7, C8, C10, C11, C14, C15, C16, C19 and C20 exhibited a fair rock quality designation. C3, C4, and C13 exhibited a poor rock quality designated. C5, C9, C12, C17, C18, C21, and C22 exhibited a good rock quality designation. Finally, C23 exhibited an excellent rock quality designation.

TH24-05: all seven (7) core runs exhibited good recovery core runs, with varying rock quality designation; C17 exhibited a very poor rock classification, followed by C18 and C19 with poor rock classification. C20 and C21 showed improvement with fair rock classification, while the final two, C22 and C23, exhibited good rock classification.

8. Frost

8.1 Seasonal Frost Penetration

The depths of frost penetration have been estimated for a range of annual air freezing identified in **Table 8-1**. The annual average freezing index was inferred from Figure K-4 of the National Building Code of Canada (2020) Commentary document. The ten-year return annual freezing index was calculated using the mean annual freezing index value and recommendations outlined in the Canadian Foundation Engineering Manual (CFEM 5e). The fifty-year return annual freezing index was taken from Figure K-5 of the National Building Code of Canada (2020) Commentary document.

Factors such as snow cover, vegetation at surface, soil type and groundwater conditions can all significantly impact the depth of frost penetration. The predominant soil type on the project site is fat clay.

Table 8-1: Frost Penetration Depth

Parameter	Period		
	Mean	10-Year Return	50-Year Return
Annual Air Freezing Index (°C-days)	1825	1875	2375
Estimated Frost Penetration (Fat Clay Subgrade) – gravel surface, no snow cover (m)	1.9	2.0	2.5
Estimated Frost Penetration (Fat Clay Subgrade) – grass with snow cover (m)	1.7	1.9	2.2

For foundation design considerations, the CFEM recommends using the ten-year return annual freezing index to predict frost penetration. It is the responsibility of the design team to select an adequate frost penetration depth to be incorporated into the design.

8.2 Frost Susceptivity

The qualitative frost susceptibility of a soil is typically assessed using guidelines developed by Casagrande (1932) based on the percentage by weight of the soil finer than 0.02 mm, and the Plasticity Index. The classification system has been adapted by the U.S. Army Corps of Engineers and the Canadian Foundation Engineering Manual (2023). Soils are classed as F1 through F4 in order of increasing frost susceptibility.

The soils (clay and silt) encountered during the geotechnical investigation fall mostly within the frost groups F3 and F4. The F3 group has high to very high susceptibility to frost and F4 has very high susceptibility. Frost susceptibility has been assigned to the encountered soil type and is summarized in **Table 8-2**.

Table 8-2: Frost Susceptibility

Soil Unit	USCS Soil Type	Frost Group	Percentage finer than 0.02 mm, by weight	PI	Frost Susceptibility
Clay/Clay fill	CL, CH	F3	-	>12	High to very high susceptibility
Silt	ML	F4	-	-	Very high susceptibility

Source: *Canadian Foundation Engineering Manual (CFEM, 5e), Chapter 14 Frost Action*

9. Seismic Considerations

As per the CFEM, the site classification for seismic site response is dependent on the average properties in the top 30 m of the soil profile. Based on a soil profile having more than 3 m of high plasticity clay and Article 4.1.8.4 of the National Building Code of Canada (NBCC) 2020, a Seismic Site Class E can be assigned to the site.

The 2020 National Building Code of Canada (NBCC) Seismic Hazard Calculation for the site is provided in **Appendix 5.** It includes values of spectral acceleration (for time periods of 0.2, 0.5, 1.0, 2.0, 5.0 and 10.0 seconds), peak ground acceleration, and peak ground velocity for 2%, 5%, and 10% probability of exceedance in 50 years.

10. References

AECOM Canada Ltd. (2021). *Technical Memorandum: High Risk River Crossings – Phase 3 – Geotechnical Condition Assessment. Winnipeg: AECOM Canada Ltd. (2021).*

AECOM Canada Ltd. (2018). *Technical Memorandum: High Risk River Crossings – Phase 2 – Geotechnical Assessment for Site 5 and 6. Winnipeg: AECOM Canada Ltd. (2018).*

Bezys, R. K., Bamburak, J. D., & Conley, G. G. (2002). *Bedrock Mineral Resources of Manitoba's Capital Region. Winnipeg: Manitoba Geological Survey*.

Canadian Commission on Building and Fire Codes, (2020). *National Building Code of Canada (NBCC) 2020*. National Research Council of Canada 2022.

Canadian Geological Society. (2023). *Canadian Foundation Engineering Manual 5th Edition*.

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American Society for Testing and Materials, (1995). *D2938 - Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens.*

Appendix 1

Site Photos

TH24-01 Drilling

TH24-01 Standpipe

TH24-02 Drilling

TH24-03 Barge Launch

TH24-03 Barge Drilling

TH24-03 Barge Demobilization

TH24-04 Drilling

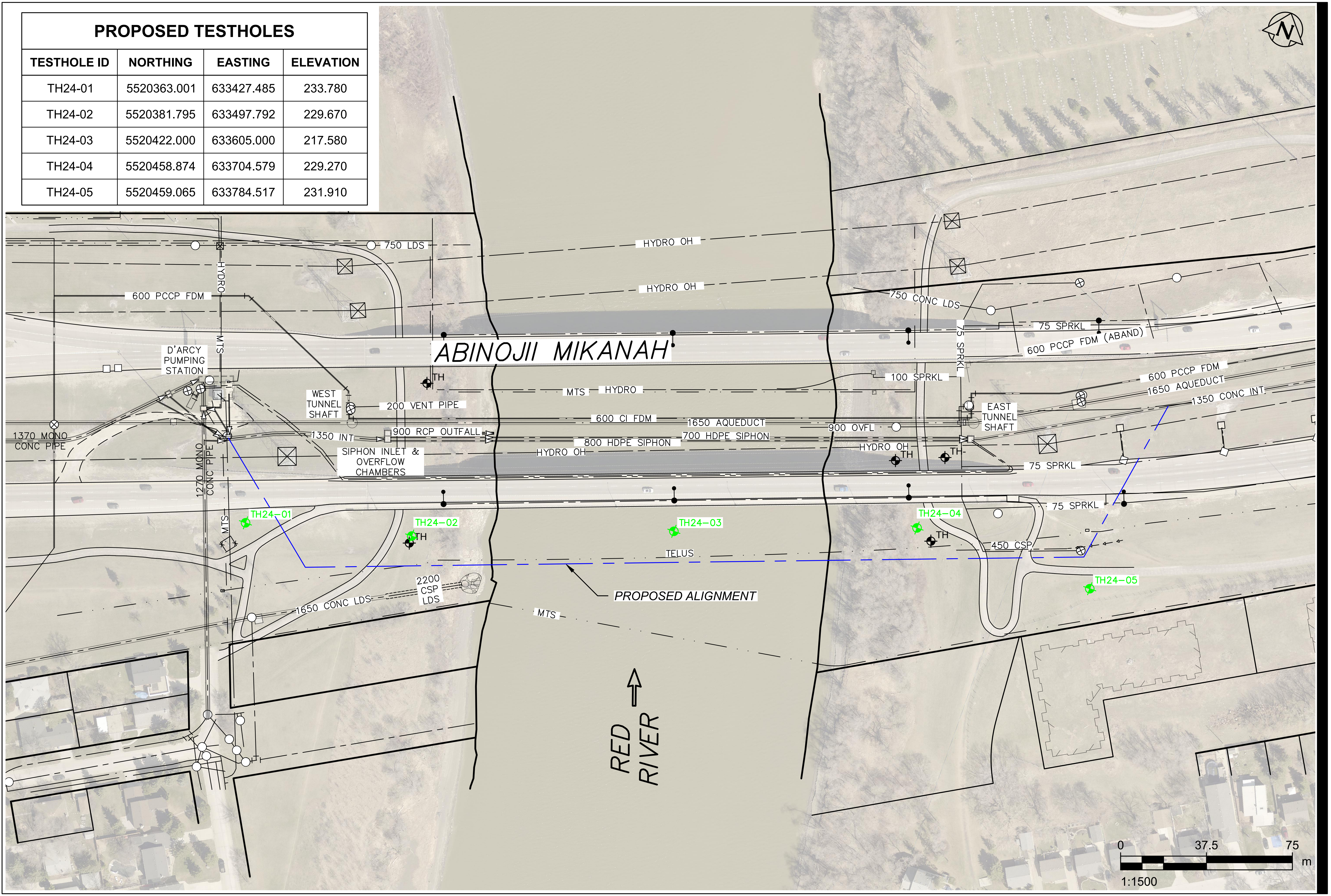
TH24-05 Standpipe

Appendix 2

Testhole Location Plan

PROPOSED TESTHOLES

TESTHOLE ID	NORTHING	EASTING	ELEVATION
TH24-01	5520363.001	633427.485	233.780
TH24-02	5520381.795	633497.792	229.670
TH24-03	5520422.000	633605.000	217.580
TH24-04	5520458.874	633704.579	229.270
TH24-05	5520459.065	633784.517	231.910



Last saved by: LEIPPIA(2024-06-11) Last Plotted: 2024-08-22
Filename: C:\USERS\LEIPPIA\DC\ACCDOCS\AECOM\60728226-REPLC-OF-FGSV-SIPHON\PROJECT FILES\900 DESIGN COLLABORATION\20 DETAILED DESIGN\B\SKE\60728226-SKE-B-1000.DWG
Project Management Initials: Designer: ____ Checked: ____ Approved: ____ ANSI B 279.4mm x 431.8mm

PROPOSED TESTHOLE
LAYOUT PLAN

Replacement of the Fort Gary St Vital Siphon
The City of Winnipeg
Water and Waste Department Engineering Division
Project No.: 60728226 Date: 2024/08/22

AECOM

Figure: 1.0

Appendix 3

Testhole Logs

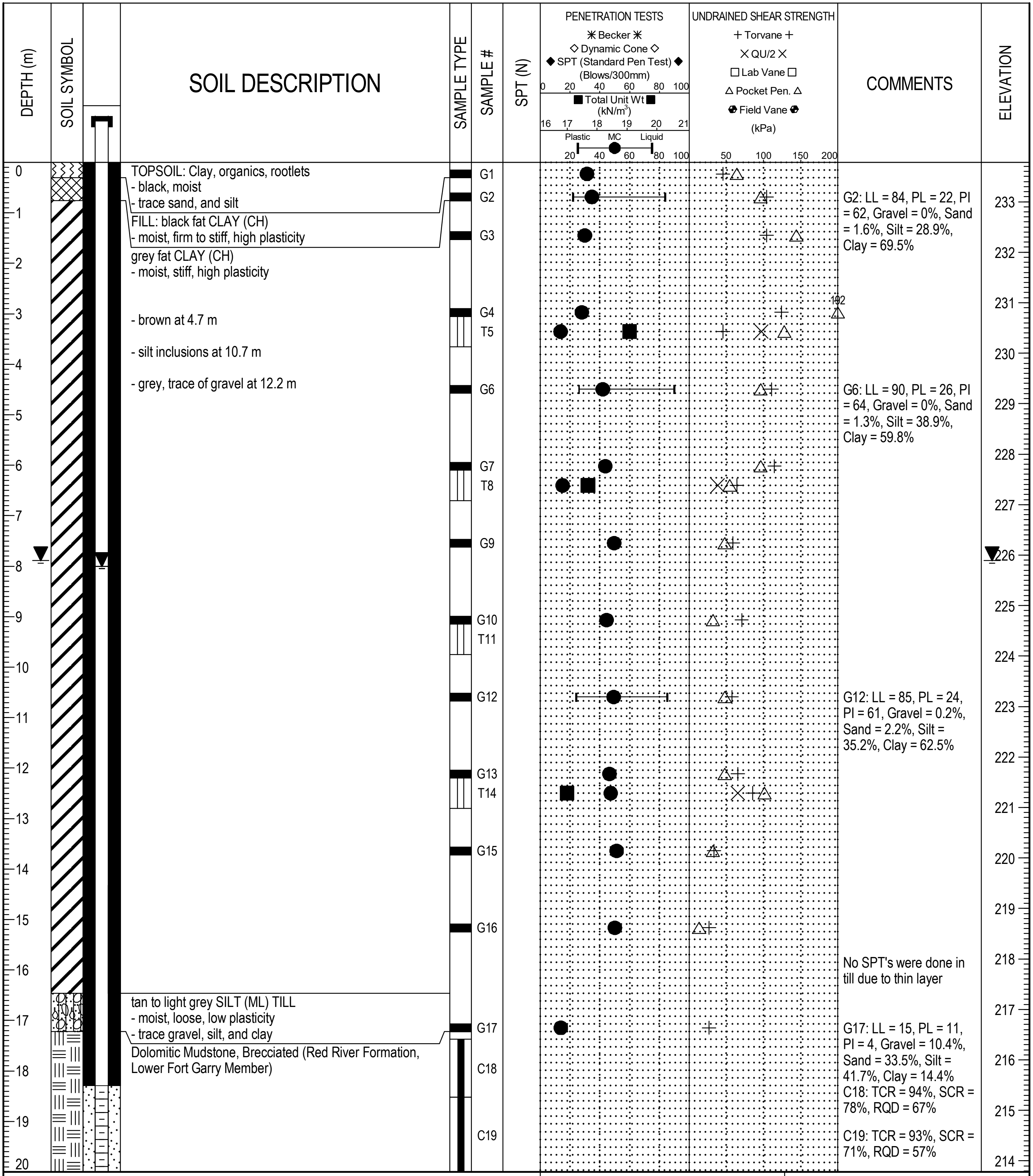
PROJECT: Replacement of the FGSV Siphon | CLIENT: City of Winnipeg | TESTHOLE NO: **TH24-01**

LOCATION: Fort Garry Bridge, Winnipeg, MB, 14 U 633427.485 m E 5520363.001 m N | PROJECT NO.: 60728226

CONTRACTOR: Paddock Drilling | METHOD: SSA/HAS | ELEVATION (m): 233.78

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



SOIL DESCRIPTION

TOPSOIL: Clay, organics, rootlets
- black, moist
- trace sand, and silt

FILL: black fat CLAY (CH)
- moist, firm to stiff, high plasticity

grey fat CLAY (CH)
- moist, stiff, high plasticity

- brown at 4.7 m

- silt inclusions at 10.7 m

- grey, trace of gravel at 12.2 m

tan to light grey SILT (ML) TILL
- moist, loose, low plasticity
- trace gravel, silt, and clay

Dolomitic Mudstone, Brecciated (Red River Formation, Lower Fort Garry Member)

COMMENTS

G2: LL = 84, PL = 22, PI = 62, Gravel = 0%, Sand = 1.6%, Silt = 28.9%, Clay = 69.5%

G6: LL = 90, PL = 26, PI = 64, Gravel = 0%, Sand = 1.3%, Silt = 38.9%, Clay = 59.8%

G12: LL = 85, PL = 24, PI = 61, Gravel = 0.2%, Sand = 2.2%, Silt = 35.2%, Clay = 62.5%

No SPT's were done in till due to thin layer

G17: LL = 15, PL = 11, PI = 4, Gravel = 10.4%, Sand = 33.5%, Silt = 41.7%, Clay = 14.4%

C18: TCR = 94%, SCR = 78%, RQD = 67%

C19: TCR = 93%, SCR = 71%, RQD = 57%

AECOM

LOGGED BY: GA | COMPLETION DEPTH: 26.14 m

REVIEWED BY: GL | COMPLETION DATE: 6/3/24

PROJECT ENGINEER: German Leal

PROJECT: Replacement of the FGSV Siphon
CLIENT: City of Winnipeg
TESTHOLE NO: **TH24-01**
LOCATION: Fort Garry Bridge, Winnipeg, MB, 14 U 633427.485 m E 5520363.001 m N
PROJECT NO.: 60728226
CONTRACTOR: Paddock Drilling
METHOD: SSA/HAS
ELEVATION (m): 233.78

SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ▨ NO RECOVERY | ▮ CORE

BACKFILL TYPE: ■ BENTONITE | ⁘ GRAVEL | ▥ SLOUGH | ▩ GROUT | ▨ CUTTINGS | ⁘ SAND

PENETRATION TESTS: ✳ Becker ✳, ◇ Dynamic Cone ◇, ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100; ■ Total Unit Wt ■ (kN/m³) 16 17 18 19 20 21; Plastic MC Liquid 20 40 60 80 100

UNDRAINED SHEAR STRENGTH: + Torvane +, × QU/2 ×, □ Lab Vane □, △ Pocket Pen. △, ● Field Vane ● (kPa) 50 100 150 200

DEPTH (m)	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
20–21.5	CORE	C20		C20: TCR = 79%, SCR = 22%, RQD = 20%	213
21.5–23.1	CORE	C21		C21: TCR = 97%, SCR = 79%, RQD = 78%	211
23.1–24.6	CORE	C22		C22: TCR = 84%, SCR = 54%, RQD = 45%	210
24.6–26.1	CORE	C23		C23: TCR = 81%, SCR = 76%, RQD = 68%	208

SOIL DESCRIPTION

END OF TEST HOLE
- Teshole terminated at depth of 26.1 m in bedrock.
- No seepage was observed due to use to coring methods.
- Groundwater level was observed at a depth of 7.9 m upon completion of drilling.
- Soil sloughing was observed below a deptht of 14.3 m.

Monitoring Well:
- Standpipe piezometer installed to a depth of 25.2 m, in bedrock, slotted between a depth of 18.3 and 25.2 m, stick up 0.9 m.
- Testhole backfilled with filter sand at 17.4 m, then with bentonite pellets to ground surface.

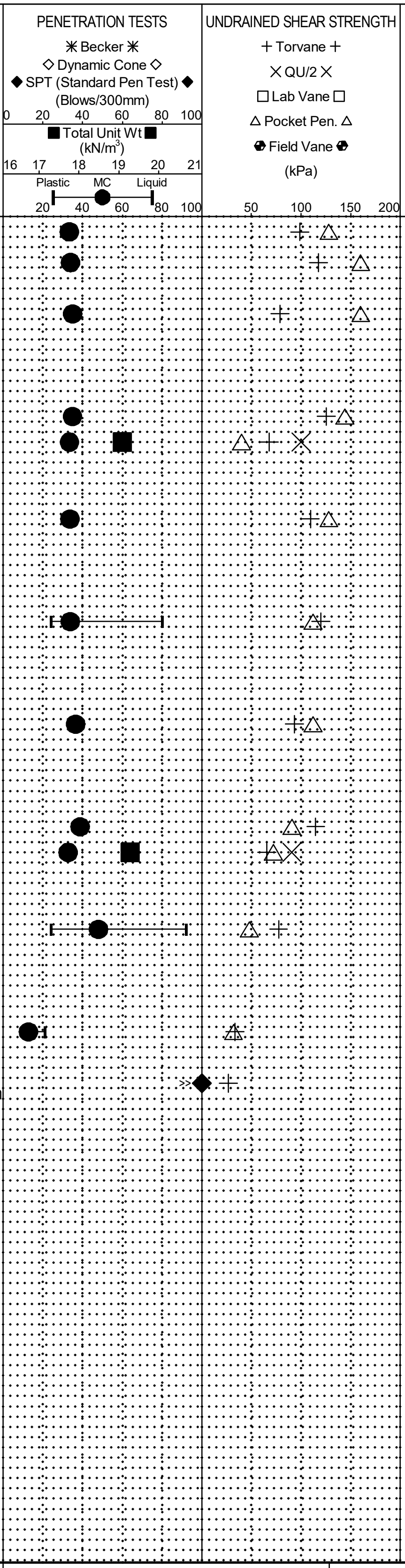
AECOM

LOGGED BY: GA
COMPLETION DEPTH: 26.14 m
REVIEWED BY: GL
COMPLETION DATE: 6/3/24
PROJECT ENGINEER: German Leal

PROJECT: Replacement of the FGSV Siphon
CLIENT: City of Winnipeg
TESTHOLE NO: **TH24-02**
LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633497.792 m E 5520381.795 m N
PROJECT NO.: 60728226
CONTRACTOR: Paddock Drilling
METHOD: SSA
ELEVATION (m): 229.67

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◪ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
0	TOPSOIL: Clay, organics, rootlets - black, moist	G1		
1	FILL: black fat CLAY (CH) - moist, stiff, high plasticity	G2		
	grey fat CLAY (CH) - moist, stiff, high plasticity - trace gravel - brown at 2.3 m - grey at 6.1 m	G3 G4 T5 G6 G7 T8 G9 G10 T11 G12		G7: LL = 80, PL = 24, PI = 56, Gravel = 0%, Sand = 1.4%, Silt = 50.4%, Clay = 48.1% G12: LL = 92, PL = 24, PI = 68, Gravel = 0%, Sand = 0.2%, Silt = 32.1%, Clay = 67.8%
	tan to light grey SILT (ML) TILL - moist, loose, low plasticity - trace silt, trace clay, and trace gravel	G13	12/ 152mm	G13: LL = 21, PL = 12, PI = 9, Gravel = 4.6%, Sand = 33.6%, Silt = 43.6%, Clay = 18.1%
13	END OF TESTHOLE - Testhole terminated at a depth of 13.0 m on suspected bedrock. - Heavy groundwater seepage was observed at a depth of 10.4 m. - Groundwater level was observed at a depth of 11.4 m upon completion of drilling. - Soil sloughing was observed below a depth of 11.0 m. - Testhole backfilled with bentonite to ground surface			



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
ELEVATION: 229 228 227 226 225 224 223 222 221 220 219 218 217 216 215 214 213 212 211 210

AECOM
LOGGED BY: GA
REVIEWED BY: GL
PROJECT ENGINEER: German Leal
COMPLETION DEPTH: 12.95 m
COMPLETION DATE: 6/4/24

PROJECT: Replacement of the FGSV Siphon | CLIENT: City of Winnipeg | TESTHOLE NO: **TH24-03**

LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633605 m E 5520422 m N | PROJECT NO.: 60728226

CONTRACTOR: Paddock Drilling | METHOD: HAS | ELEVATION (m): 223.98

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◩ NO RECOVERY ▌CORE

DEPTH (m)	SOIL SYMBOL	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + × QU/2 × ☐ Lab Vane ☐ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
0		Red River							
1									223
2									222
3									221
4									220
5									219
6									218
		Alluvial Deposits - Note: no samples and testing were conducted due to time constraint							
7									217
8									216
		Dolomitic Mudstone, Brecciated (Red River Formation, Lower Fort Garry Member)	▌	C1				C1: TCR = 61%, SCR = 28%, RQD = 0%	
			▌	C2				C2: TCR = 95%, SCR = 97%, RQD = 53%	
9									215
10			▌	C3				C3: TCR = 96%, SCR = 81%, RQD = 47%	214
11									213
			▌	C4				C4: TCR = 90%, SCR = 71%, RQD = 41%	
12									212
13			▌	C5				C5: TCR = 98%, SCR = 96%, RQD = 81%	211
14			▌	C6				C6: TCR = 91%, SCR = 68%, RQD = 68%	210
			▌	C7				C7: TCR = 87%, SCR = 80%, RQD = 56%	
15									209
			▌	C8				C8: TCR = 96%, SCR = 82%, RQD = 72%	
16									208
			▌	C9				C9: TCR = 94%, SCR = 88%, RQD = 86%	
17									207
			▌	C10				C10: TCR = 96%, SCR = 75%, RQD = 57%	
18									206
19			▌	C11				C11: TCR = 98%, SCR = 86%, RQD = 64%	205
20									

AECOM

LOGGED BY: GA | COMPLETION DEPTH: 35.05 m

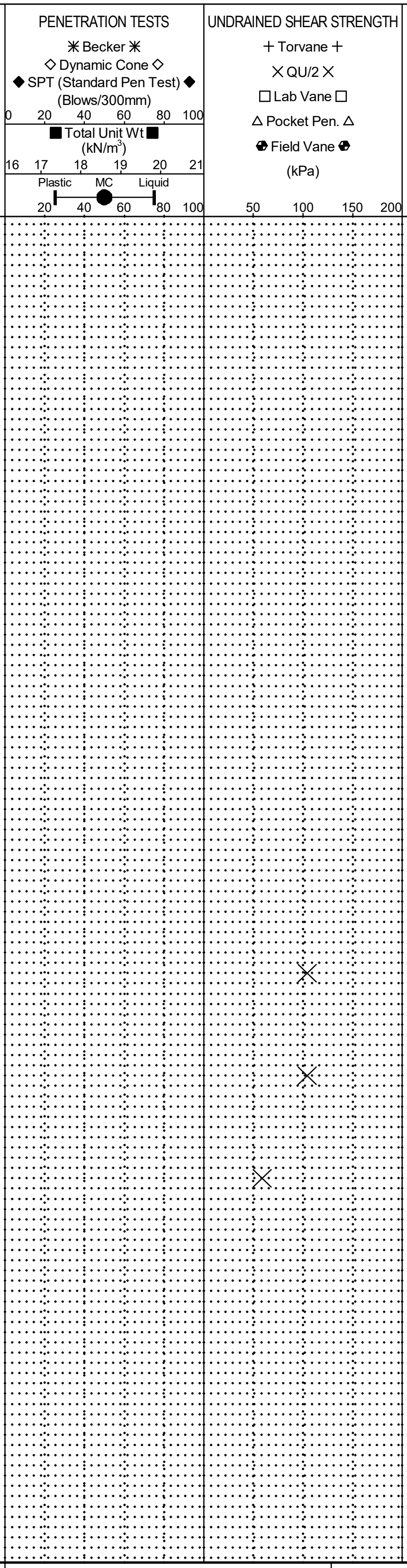
REVIEWED BY: GL | COMPLETION DATE: 8/13/24

PROJECT ENGINEER: German Leal

PROJECT: Replacement of the FGSV Siphon
CLIENT: City of Winnipeg
TESTHOLE NO: **TH24-03**
LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633605 m E 5520422 m N
PROJECT NO.: 60728226
CONTRACTOR: Paddock Drilling
METHOD: HAS
ELEVATION (m): 223.98

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
20–35		CORE	C12		C12: TCR = 91%, SCR = 88%, RQD = 84%
		CORE	C13		C13: TCR = 93%, SCR = 65%, RQD = 39%
		CORE	C14		C14: TCR = 88%, SCR = 73%, RQD = 60%
		CORE	C15		C15: TCR = 87%, SCR = 70%, RQD = 70%
		CORE	C16		C16: TCR = 92%, SCR = 66%, RQD = 62%
		CORE	C17		C17: TCR = 94%, SCR = 90%, RQD = 90%
		CORE	C18		C18: TCR = 98%, SCR = 86%, RQD = 84%
		CORE	C19		C19: TCR = 98%, SCR = 81%, RQD = 73%
		CORE	C20		C20: TCR = 97%, SCR = 70%, RQD = 59%
		CORE	C21		C21: TCR = 98%, SCR = 90%, RQD = 83%
		CORE	C22		C22: TCR = 99%, SCR = 98%, RQD = 89%
		CORE	C23		C23: TCR = 97%, SCR = 96%, RQD = 94%
35	END OF TEST HOLE - Teshole terminated at depth of 35 m in bedrock. - No seepage was observed due to use to coring methods. - No groundwater level was observed due to coring methods. - No soil sloughing was observed due to coring methods. - River level was observed at an elevation of 223.98 m.				



DEPTH (m): 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40

ELEVATION: 203, 202, 201, 200, 199, 198, 197, 196, 195, 194, 193, 192, 191, 190, 189, 188, 187, 186, 185

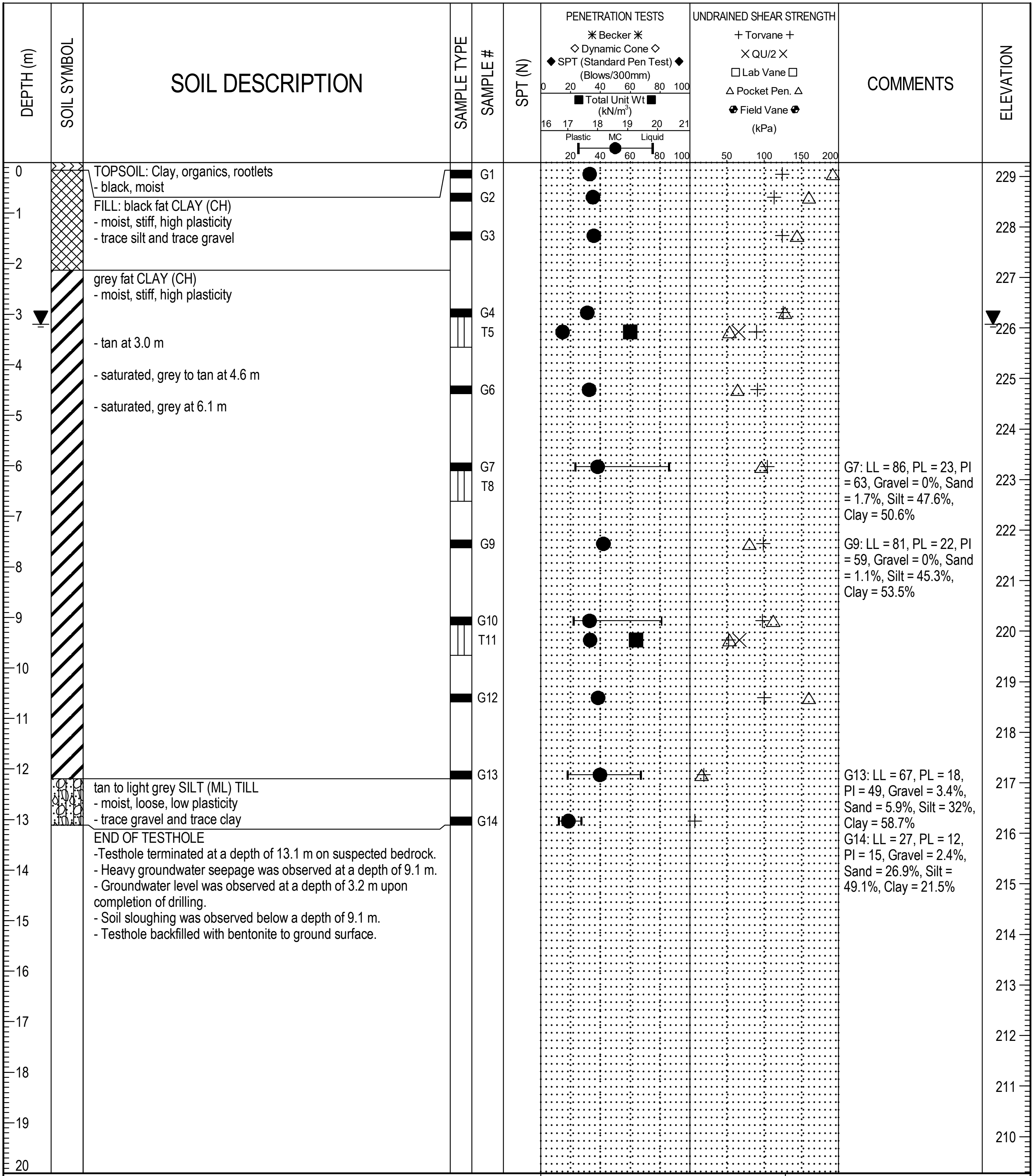
LOG OF TEST HOLE 60728226 - TESTHOLE LOGS - FGB - 20240820-GA .GPJ UMA WINN.GDT 10/4/24

AECOM

LOGGED BY: GA
REVIEWED BY: GL
PROJECT ENGINEER: German Leal
COMPLETION DEPTH: 35.05 m
COMPLETION DATE: 8/13/24

PROJECT: Replacement of the FGSV Siphon	CLIENT: City of Winnipeg	TESTHOLE NO: **TH24-04**
LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633704.579 m E 5520458.874 m N		PROJECT NO.: 60728226
CONTRACTOR: Paddock Drilling	METHOD: SSA	ELEVATION (m): 229.27

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE



SOIL DESCRIPTION

TOPSOIL: Clay, organics, rootlets
- black, moist

FILL: black fat CLAY (CH)
- moist, stiff, high plasticity
- trace silt and trace gravel

grey fat CLAY (CH)
- moist, stiff, high plasticity

- tan at 3.0 m

- saturated, grey to tan at 4.6 m

- saturated, grey at 6.1 m

tan to light grey SILT (ML) TILL
- moist, loose, low plasticity
- trace gravel and trace clay

END OF TESTHOLE
-Testhole terminated at a depth of 13.1 m on suspected bedrock.
- Heavy groundwater seepage was observed at a depth of 9.1 m.
- Groundwater level was observed at a depth of 3.2 m upon completion of drilling.
- Soil sloughing was observed below a depth of 9.1 m.
- Testhole backfilled with bentonite to ground surface.

COMMENTS

G7: LL = 86, PL = 23, PI = 63, Gravel = 0%, Sand = 1.7%, Silt = 47.6%, Clay = 50.6%

G9: LL = 81, PL = 22, PI = 59, Gravel = 0%, Sand = 1.1%, Silt = 45.3%, Clay = 53.5%

G13: LL = 67, PL = 18, PI = 49, Gravel = 3.4%, Sand = 5.9%, Silt = 32%, Clay = 58.7%
G14: LL = 27, PL = 12, PI = 15, Gravel = 2.4%, Sand = 26.9%, Silt = 49.1%, Clay = 21.5%

LOGGED BY: GA	COMPLETION DEPTH: 13.11 m
REVIEWED BY: GL	COMPLETION DATE: 6/6/24
PROJECT ENGINEER: German Leal	Page 1 of 1

PROJECT: Replacement of the FGSV Siphon | CLIENT: City of Winnipeg | TESTHOLE NO: **TH24-05**

LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633784.517 m E 5520459.065 m N | PROJECT NO.: 60728226

CONTRACTOR: Paddock Drilling | METHOD: SSA/HAS | ELEVATION (m): 231.91

SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ▨ NO RECOVERY | ▮ CORE

BACKFILL TYPE: ■ BENTONITE | ⁚ GRAVEL | ▥ SLOUGH | ⁙ GROUT | ▨ CUTTINGS | ⁚ SAND

DEPTH (m) | SOIL SYMBOL | SOIL DESCRIPTION | SAMPLE TYPE | SAMPLE # | SPT (N) | COMMENTS | ELEVATION

SOIL DESCRIPTION

TOPSOIL: Clay, organics, rootlets
- black, moist
- trace sand and trace gravel

FILL: black fat CLAY (CH)
- moist, stiff, high plasticity
- trace sand and trace gravel

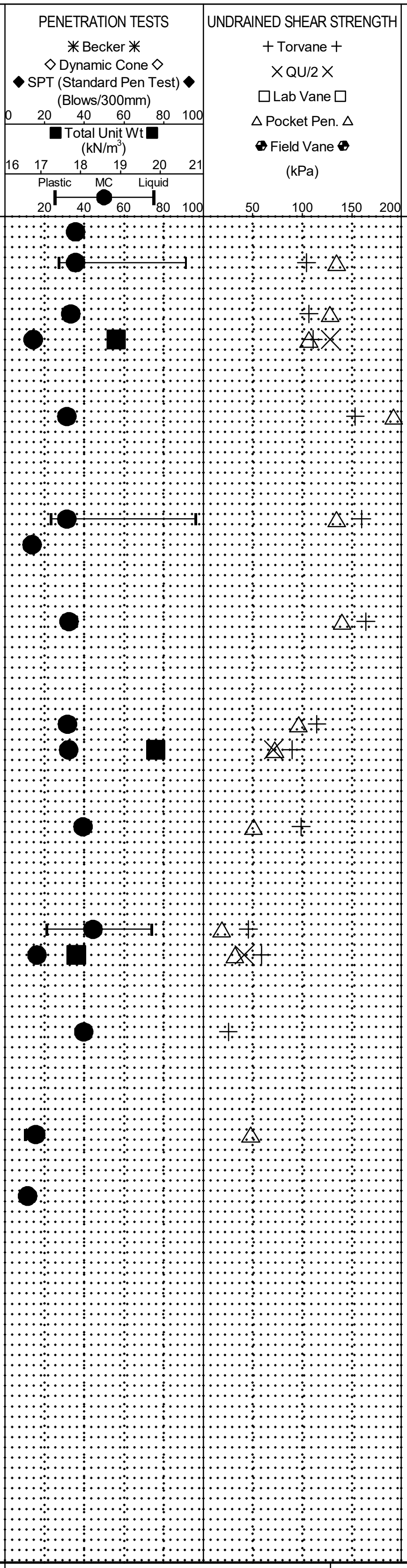
brown fat CLAY (CH)
- moist, stiff, high plasticity

- grey at 7.6 m

tan to light grey SILT (ML) TILL
- moist, loose, low plasticity
- trace sand, trace clay and trace gravel

Dolomitic Mudstone, Brecciated (Red River Formation, Lower Fort Garry Member)

SAMPLE #	COMMENTS
G1	
G2	G2: LL = 91, PL = 27, PI = 64, Gravel = 0%, Sand = 0.9%, Silt = 44.6%, Clay = 54.6%
G3	
T4	
G5	
G6	G6: LL = 96, PL = 23, PI = 73, Gravel = 0%, Sand = 0.1%, Silt = 47.8%, Clay = 52.1%
T7	
G8	
G9	
T10	
G11	
G12	G12: LL = 74, PL = 21, PI = 53, Gravel = 0.2%, Sand = 1.6%, Silt = 35%, Clay = 63.2%
T13	
G14	
G15	G15: LL = 18, PL = 10, PI = 8, Gravel = 8%, Sand = 36.8%, Silt = 38.9%, Clay = 16.2%
G16	
C17	C17: TCR = 69%, SCR = 0%, RQD = 0%
C18	C18: TCR = 78%, SCR = 30%, RQD = 25%
C19	C19: TCR = 81%, SCR = 32%, RQD = 29%
C20	C20: TCR = 94%, SCR = 32%, RQD = 29%



DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

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

LOG OF TEST HOLE 60728226 - TESTHOLE LOGS - FGB - 20240820-GA .GPJ UMA WINN.GDT 10/4/24

AECOM

LOGGED BY: GA | COMPLETION DEPTH: 14.63 m

REVIEWED BY: GL | COMPLETION DATE: 6/5/24

PROJECT ENGINEER: German Leal

PROJECT: Replacement of the FGSV Siphon
CLIENT: City of Winnipeg
TESTHOLE NO: **TH24-05**
LOCATION: Fort Garry Bridge, Winnipeg, MB 14 U 633784.517 m E 5520459.065 m N
PROJECT NO.: 60728226
CONTRACTOR: Paddock Drilling
METHOD: SSA/HAS
ELEVATION (m): 231.91

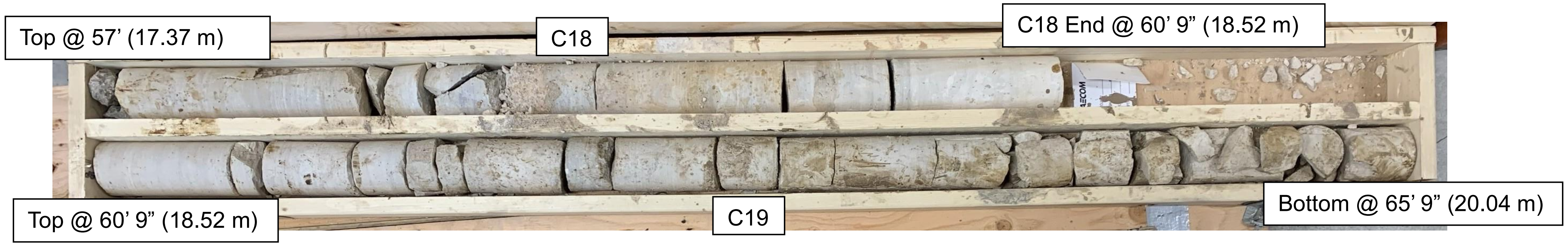
SAMPLE TYPE	■ GRAB	▥ SHELBY TUBE	☒ SPLIT SPOON	☰ BULK	▱ NO RECOVERY	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ GRAVEL	▥ SLOUGH	▩ GROUT	▨ CUTTINGS	⁘ SAND

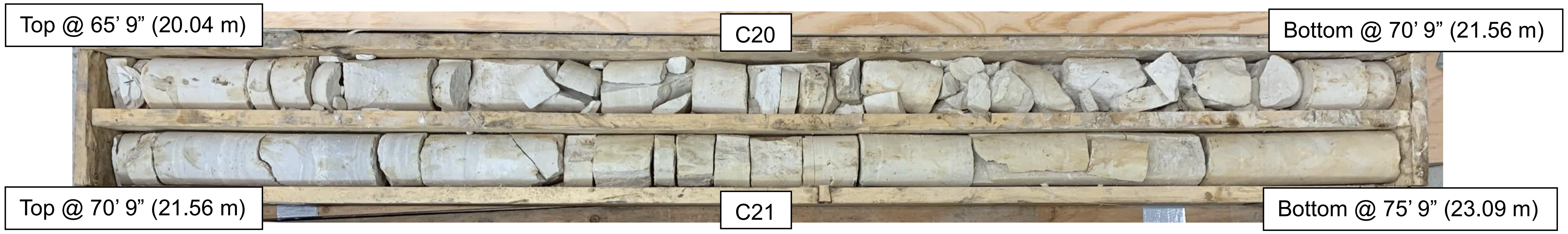
DEPTH (m)	SOIL SYMBOL	SLOTTED PIEZOMETER	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	PENETRATION TESTS ✳ Becker ✳ ◇ Dynamic Cone ◇ ◆ SPT (Standard Pen Test) ◆ (Blows/300mm) 0 20 40 60 80 100 ■ Total Unit Wt ■ (kN/m^3) 16 17 18 19 20 21 Plastic MC Liquid 20 40 60 80 100	UNDRAINED SHEAR STRENGTH + Torvane + ✕ QU/2 ✕ □ Lab Vane □ △ Pocket Pen. △ ● Field Vane ● (kPa) 50 100 150 200	COMMENTS	ELEVATION
20				▮						
21				▮	C21				C21: TCR = 92%, SCR = 70%, RQD = 62%	211
22				▮						210
23				▮	C22				C22: TCR = 96%, SCR = 88%, RQD = 87%	209
24				▮	C23				C23: TCR = 89%, SCR = 85%, RQD = 80%	208
25			END OF TEST HOLE - Teshole terminated at depth of 24.7 m in bedrock. - No seepage was observed due to use to coring methods. - Groundwater level was observed at a depth of 5.1 m upon completion of drilling. - No soil sloughing was observed during or upong completion of drilling. Monitoring Well: - Standpipe piezometer installed to a depth of 24.7 m, in bedrock, slotted between a depth of 24.7 m and 15.5 m, stick up 0.9 m. - Testhole backfilled with filter sand, then with bentonite pellets to ground surface.							207
26										206
27										205
28										204
29										203
30										202
31										201
32										200
33										199
34										198
35										197
36										196
37										195
38										194
39										193
40										

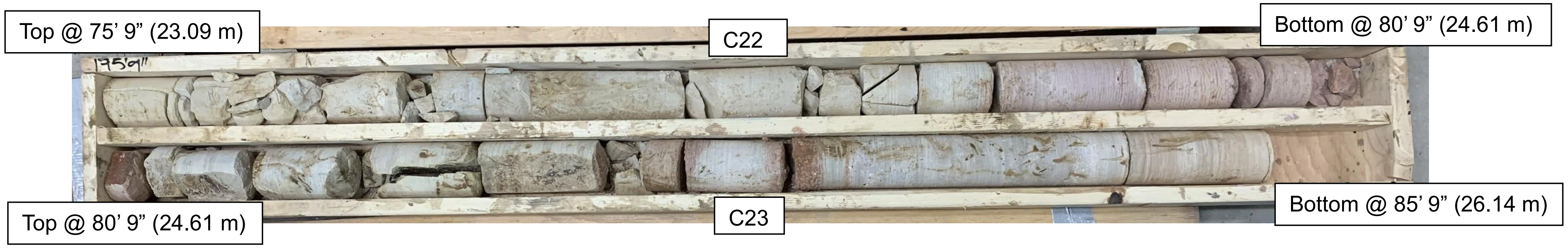
LOGGED BY: GA
COMPLETION DEPTH: 14.63 m
REVIEWED BY: GL
COMPLETION DATE: 6/5/24
PROJECT ENGINEER: German Leal

Replacement of the FGSV Siphon Crossing the Red River

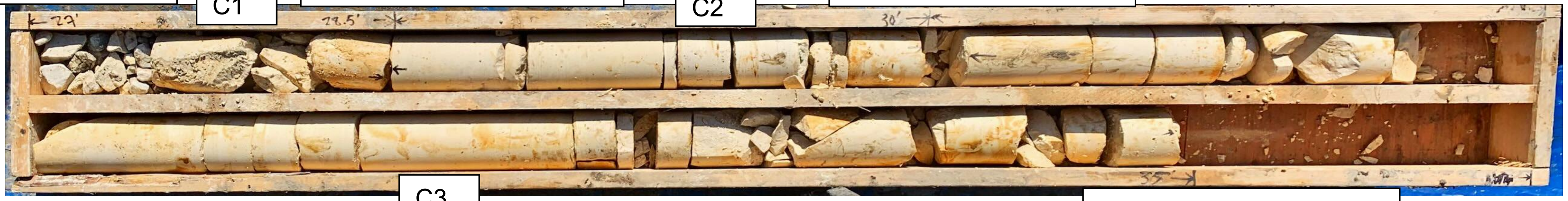
TH24-01 Core Runs







TH24-03 Core Runs



Top @ 27 (8.23 m)

C1

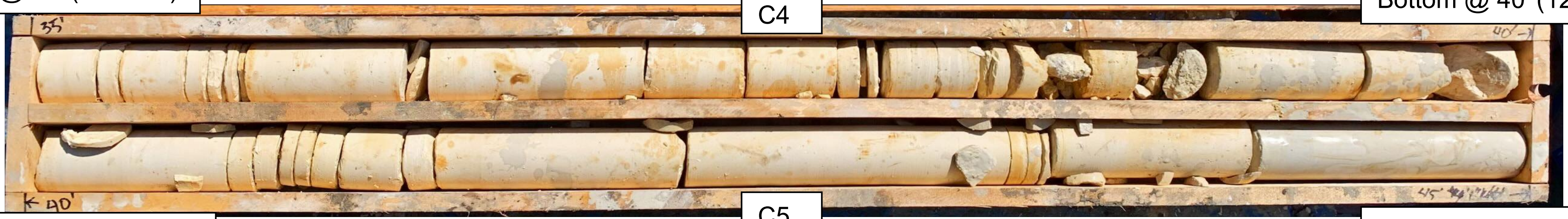
C1 Ends @ 28.5’ (8.69 m)

C2

C2 Ends @ 30’ (9.14 m)

C3

Bottom @ 35’ (10.67 m)



Top @ 35’ (10.67 m)

C4

Bottom @ 40’ (12.19 m)

Top @ 40’ (12.19 m)

C5

Bottom @ 45’ (13.72 m)

Top @ 45’ (13.72 m)

C6

C6 Ends @ 46’ 10” (14.22 m)

C7

Bottom @ 50’ (15.24 m)

Top @ 50’ (15.24 m)

C8

C8 Ends @ 52’ (15.85 m)

C9

Bottom @ 55’ (16.76 m)

Top @ 55’ (16.76 m)

C10

Bottom @ 60’ (18.29 m)

Top @ 60’ (18.29 m)

C11

Bottom @ 65’ (19.81 m)

Top @ 65' (19.81 m)

C12

C12 Ends @ 68' 8" (20.93 m)

C13

Bottom @ 70' (21.34 m)

Top @ 70' (21.34 m)

C14

Bottom @ 75' (22.86 m)

Top @ 75' (22.86 m)

C15

C15 Ends @ 78' 6" (23.93 m)

C16

C16 Ends @ 82.5' (25.15 m)

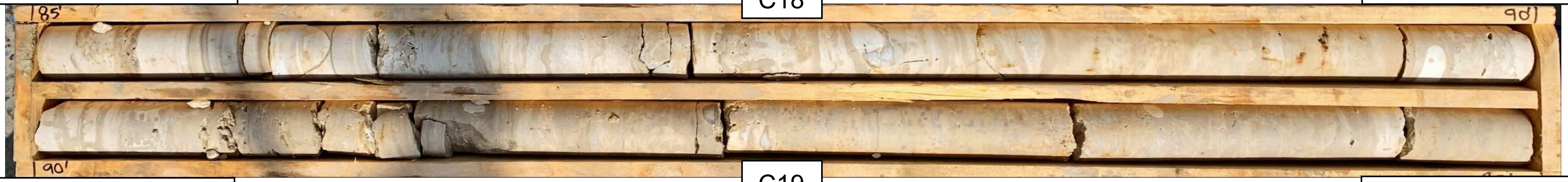
C17

Bottom @ 85' (25.91 m)

Top @ 85' (25.91 m)

C18

Bottom @ 90' (27.43 m)



Top @ 90' (27.43 m)

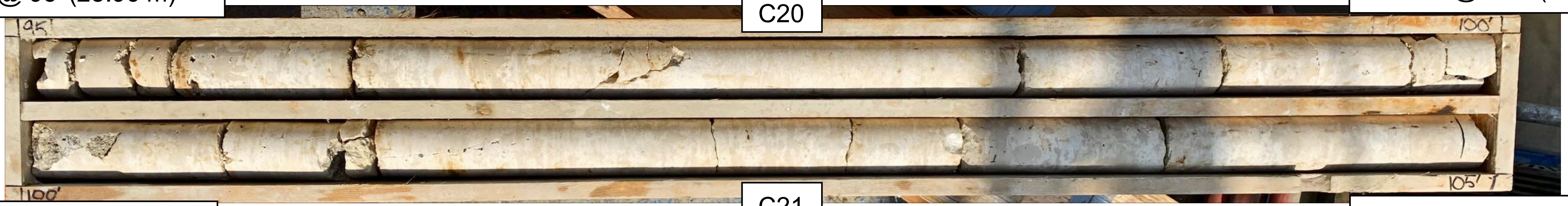
C19

Bottom @ 95' (28.96 m)

Top @ 95' (28.96 m)

C20

Bottom @ 100' (30.48 m)



Top @ 100' (30.48 m)

C21

Bottom @ 105' (32.00 m)

Top @ 105' (32.00 m)

C22

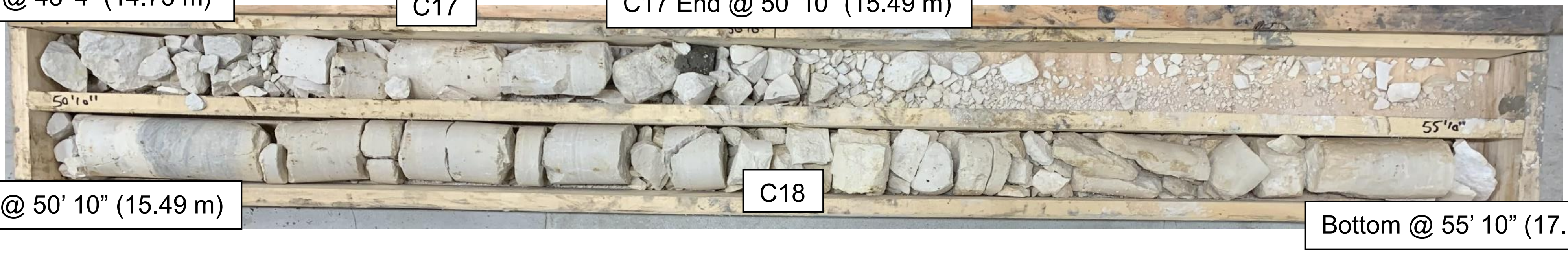
Bottom @ 110' (33.53 m)

C23

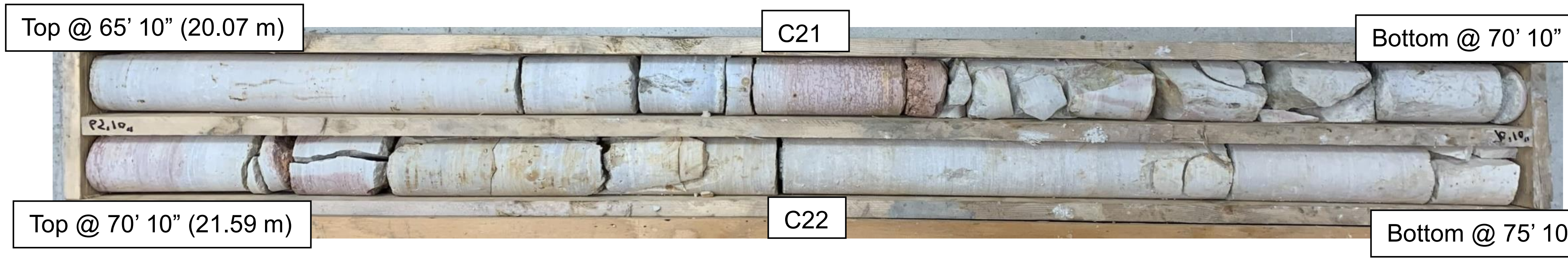
Top @ 110' (33.53 m)

Bottom @ 115' (35.05 m)

TH24-05 Core Runs


Top @ 48’ 4” (14.73 m)
C17
C17 End @ 50’ 10” (15.49 m)
Top @ 50’ 10” (15.49 m)
C18
Bottom @ 55’ 10” (17.02 m)


Top @ 55’ 10” (17.02 m)
C19
Bottom @ 60’ 10” (18.54 m)
Top @ 60’ 10” (18.54 m)
C20
Bottom @ 65’ 10” (20.07 m)


Top @ 65’ 10” (20.07 m)
C21
Bottom @ 70’ 10” (21.59 m)
Top @ 70’ 10” (21.59 m)
C22
Bottom @ 75’ 10” (23.11 m)


Top @ 75’ 10” (23.11 m)
C23
Bottom @ 81’ (23.11 m)

EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results, as shown for each hole, are described below.

1. EXPLANATION OF SOIL

Each soil stratum is classified and described noting any special conditions. The Modified Unified Classification System (MUCS) is used. The soil profile refers to the existing ground level at the time the hole was done. Where available, the ground elevation is shown. The soil symbols used are shown in detail on the soil classification chart.

1.1 Tests on Soil Samples

Laboratory and field tests are identified by the following and are on the logs:

γ_D - Dry Unit Weight. Usually expressed in kN/m^3.

γ_T - Total (moist, wet, or bulk) Unit Weight. Usually expressed in kN/m^3.

C_U - Undrained Shear Strength. Usually expressed in kPa. This value can be determined by a field vane shear test and may also be used in determining the allowable bearing capacity of the soil.

C_{PEN} - Pocket Penetrometer Reading. Usually expressed in kPa. Estimate of the undrained shear strength as determined by a pocket penetrometer.

N - Standard Penetration Test (SPT) Blow Count. The SPT is conducted in the field to assess the in-situ consistency of cohesive soils and the relative density of non-cohesive soils. The N value recorded is the number of blows from a 63.5 kg hammer free falling of 760 mm (30 in.) which is required to drive a 50 mm (2 in.) split spoon sampler 300 mm (12 in.) into the soil.

Q_U - Unconfined Compressive Strength. Usually expressed in kPa and may be used in determining allowable bearing capacity of the soil.

The following tests may also be performed on selected soil samples and the results are given on separate sheets enclosed with the logs:

- Grain Size Analysis
- Standard or Modified Proctor Compaction Test
- California Bearing Ratio Test
- Direct Shear Test
- Permeability Test
- Consolidation Test
- Triaxial Test

1.2 Natural Moisture Content

The relationship between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits for a sample should be compared to its natural moisture content and plotted on the Plasticity Chart to determine the soil classification.

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

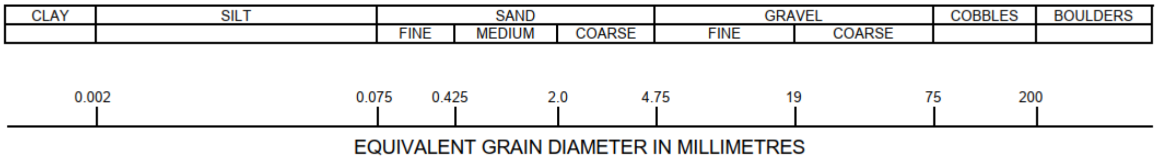
1.3 Grian Size Distrubtion

Laboratory grain size analyses provided by AECOM follow the following system. Note that, with the exception of those samples where a grain size distribution analysis has been completed, all samples have been classified by visual inspection. Visual inspection classification is not sufficient to provide exact gain sizing.

SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 – 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	ADJECTIVE
	MEDIUM	2.00	0.425		
	FINE	0.425	0.075	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.075		10 – 1	TRACE
OVERSIZE MATERIALS					
ROUNDED OR SUB-ROUNDED COBBLES 75 mm TO 200 mm BOULDERS >200 mm			ANGULAR ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME		

ISSMFE / USCS SOIL CLASSIFICATION



1.4 Soil Compactness and Consistency

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by in-situ vane tests, penetrometer tests, unconfined compression tests, or similar field and laboratory analysis. Standard Penetration Test ‘N’ values can also be used to provide an approximate indication of the consistency and shear strength of fine-grained, cohesive soils.

The standard terminology to describe cohesionless soils includes the compactness condition as determined by the Standard Penetration Test ‘N’ value. These approximate relationships are summarized in the following tables:

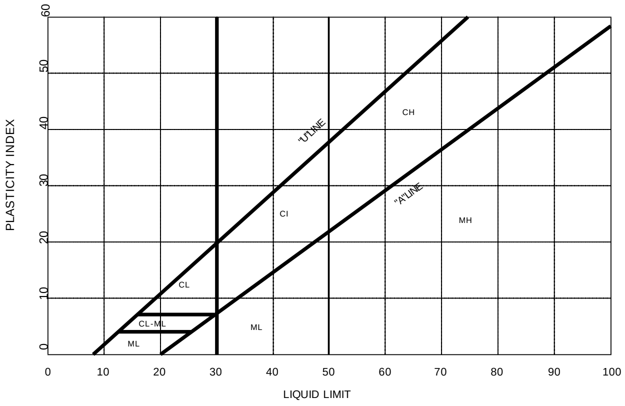
Table 1 Cohesive Soils

Consistency	SPT N (blows/0.3m)	C_u (kPa) approx.
Very Soft	<2	<12
Soft	2 - 4	12 - 25
Firm	4 - 8	25 - 50
Stiff	8 - 15	50 - 100
Very Stiff	15 - 30	100 - 200
Hard	>30	>200

Table 2 Cohesionless Soils

Compactness Condition	SPT N (blows/0.3m)
Very Loose	0 – 4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	>50

MAJOR DIVISION			UCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4 \quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{to} 3$	
			GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW ‘A’ LINE W_p LESS THAN 4
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE ‘A’ LINE W_p MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE R NO FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6 \quad C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{to} 3$	
			SP	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW ‘A’ LINE W_p LESS THAN 4
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE ‘A’ LINE W_p MORE THAN 7
FINE GRAINED SOILS	SILTS (BELOW ‘A’ LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L > 50$	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER ‘F’. E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
	CLAYS (ABOVE ‘A’ LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$	CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30 < W_L < 50$	CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50$	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW ‘A’ LINE)	$W_L < 50$	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50$	OH	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE	
BEDROCK			BR	SEE REPORT DESCRIPTION		
FILL			FILL	SEE REPORT DESCRIPTION		



NOTE:
1. BOUNDARY CLASSIFICATION POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%

SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 – 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	_____Y
	MEDIUM	2.00	0.425		
	FINE	0.425	0.075	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.075		10 – 1	TRACE

OVERSIZE MATERIALS

ROUNDED OR SUB-ROUNDED COBBLES 75 mm TO 200 mm BOULDERS >200 mm	ANGULAR ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME

MODIFIED UNIFIED SOIL
CLASSIFICATION SYSTEM

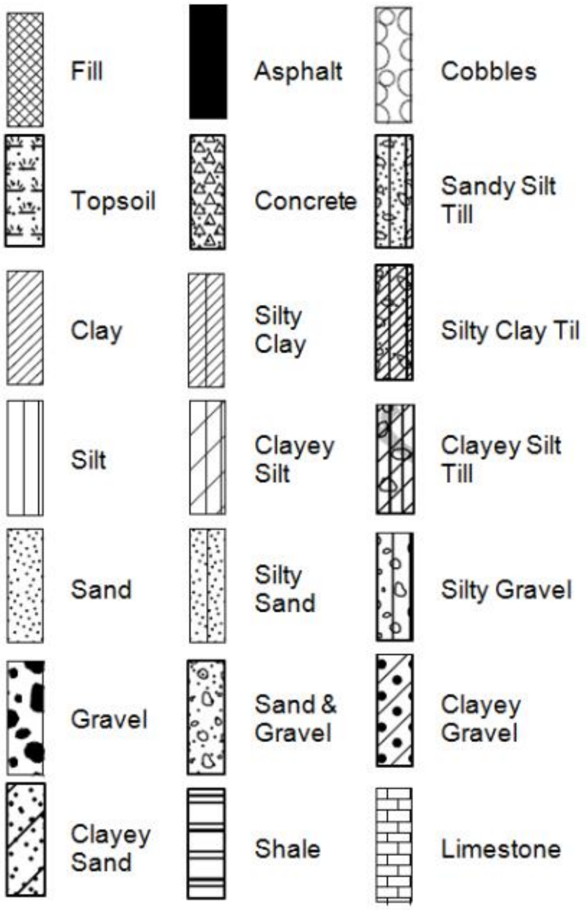
February 2022

1.5 Sample Type, Symbols and Abbreviations

The depth, type, and condition of samples are indicated on the logs by the following symbols or abbreviations:

Sample abbreviations:	Symbols:
GS: Grab Sample	■ Grab ☰ Bulk ◪ No Recovery ▥ Shelby Tube ☒ Split Spoon ◨ Core Sample
BK: Bulk Sample	
NR: No Recovery	
ST: Shelby Tube	
SS: Split Spoon	
Core: Core Samples	
FV: Field Vane	
PP: Pocket Penetrometer	
DCPT: Dynamic cone penetration test	

1.6 STRATA/Graphic Plot (Shall be Changed For Different Guidelines)



2. EXPLANATION OF ENVIROMENTAL SAMPLE

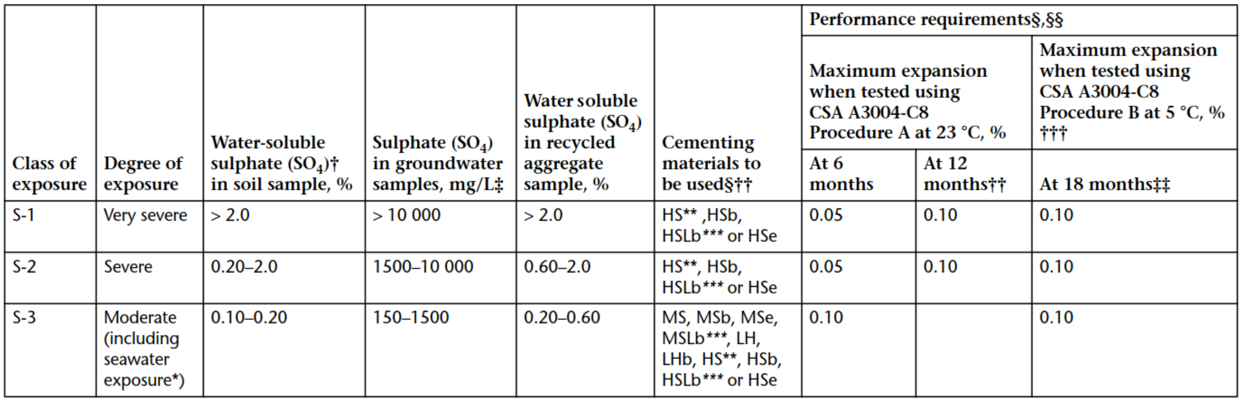
2.1 Contaminant Abbreviations

Contaminant Abbreviations	
BNAE	Base/neutral/acid extractables
BTEX	Benzene, toluene, ethylbenzene, xylenes
OCP	Organochlorine pesticides
MI	Metals and inorganics
PAH	Polycyclic aromatic hydrocarbons
PCB	Polychlorinated biphenyls
PHC	CCME petroleum hydrocarbons (fractions 1-4)
VOC	Volatile organic compounds (includes BTEX)
SO_4	Water Soluble Sulphate Content

2.2 Water Soluble Sulphate Concentration

The following table, from CSA Standard A23.1-14, indicates the requirements for concrete subjected to sulphate attack based upon the percentage of water-soluble sulphate as presented on the logs. CSA Standard A23.1-14 should be read in conjunction with the table.

Table 3 Requirements for Concrete Subjected to Sulphate Attack*



						Performance requirements§,§§		
						Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, %		Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % †††
Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	At 6 months	At 12 months††	At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

*For sea water exposure, also see Clause 4.1.1.5.

†In accordance with CSA A23.2-3B.

‡In accordance with CSA A23.2-2B.

§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).

**Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.

††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.

‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.

§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

***Where MSLb or HSLb cements are proposed for use, or where MSe or HSe combinations include Portland-limestone cement, they must also contain a minimum of 25% Type F fly ash or 40% slag or 15% metakaolin (meeting Type N pozzolan requirements) or a combination of 5% Type SF silica fume with 25% slag or a combination of 5% Type SF silica fume with 20% Type F fly ash. For some proposed MSLb, HSLb, and MSe or HSe combinations that include Portland-limestone cement, higher SCM replacement levels may be required to meet the A3004-C8 Procedure B expansion limits. Due to the 18-month test period, SCM replacements higher than the identified minimum levels should also be tested. In addition, sulphate resistance testing shall be run on MSLb and HSLb cement and MSe or HSe combinations that include Portland-limestone cement at both 23 °C and 5 °C as specified in the table.

†††If the expansion is greater than 0.05% at 6 months but less than 0.10% at 1 year, the cementing materials combination under test shall be considered to have passed.

2.3 Soil Corrosivity

The following table, from the Handbook of Corrosion Engineering (Roberge, 1999) indicates the corrosivity rating can be obtained from the soil resistivity, presented on the logs.

Table 4 Corrosivity Ratings Based on Soil Resistivity

Soil Resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

3. HYDROGEOLOGICAL

The groundwater table is indicated by the equilibrium level of water in a standpipe installed in a test hole or test pit. This level is generally taken at least 24 hours after installation of the standpipe. The groundwater level is subject to seasonal variations and is usually highest in the spring. The symbol on the logs indicating the groundwater level is an inverted solid triangle (▼).

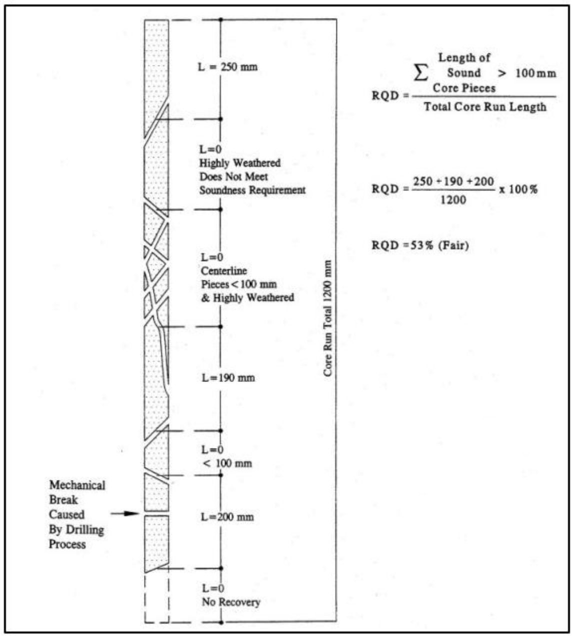
4. EXPLANATION OF ROCK

4.1 General Description and Terms

General Description of Geotechnical Unit including: Quantitative description including rock type (s), percentage of rock types, frequency and sizes of interbeds, colour, texture, weathering, strength and general joint spacing

Total Core Recovery (TCR): Total length of core recovered expressed as percentage of core run length.
Solid Core Recovery (SCR): Total length of solid full diameter core expressed as percentage of core run length.
Rock Quality Designation (RQD): Sum of lengths of solid core pieces longer than 100 mm expressed as percentage of core run length.
Fracture Index (FI): Number of fractures per meter of core.

4.2 Rock Quality Designation (RQD)

RQD(%)	RQD Classification	
0 – 25	Very Poor Quality	
25 – 50	Poor Quality	
50 – 75	Fair Quality	
75 – 90	Good Quality	
90 – 100	Excellent Quality	

4.3 Classification of Strength

Grade	Description	Field identification	Approximate range of Uniaxial compression strength (MPa)
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0

R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

4.4 Classification of Weathering

Grade	Description	Field identification
W1	Fresh	No visible sign of rock material weathering; perhaps slight discolouration on major discontinuity surface
W2	Slightly Weathered	Discolouration indicates weathering of rock material and discontinuity surface. All the rock material may be discoloured by weathering and may be somewhat weaker externally than in its fresh condition
W3	Moderately Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
W4	Highly Weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones.
W5	Completely Weathered	All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact. All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but soil has not been significantly transported.
W6	Residual Soil	Residual Soil

4.5 Type of discontinuity

Symbol	Description
F	Fault
J	Joint
Sh	Shear
Fo	Foliation
V	Vein
B	Bedding

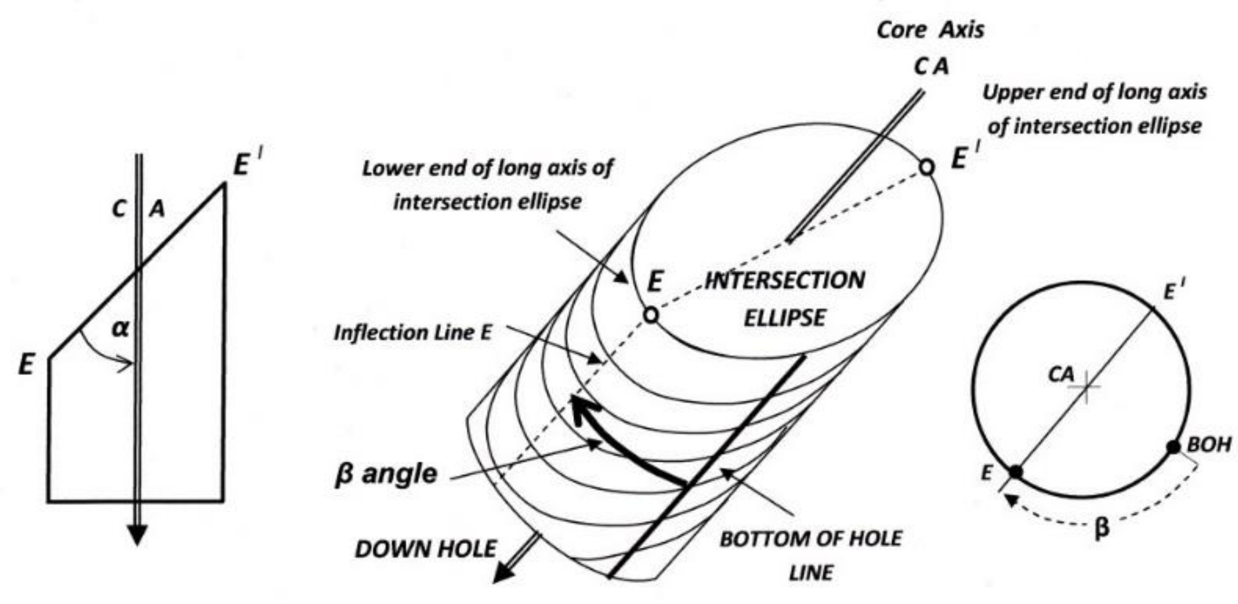
4.6 Spacing of discontinuity

Spacing Classification	Spacing width
Extremely close	<0.02m

Very close	0.02-0.06m
Close	0.06-0.2m
Moderately Close	0.2-0.6m
Wide	0.6-2.0m
Very Wide	2.0-6.0m
Extremely Wide	>6.0m

4.7 Joint Orientation

The orientation of a planar surface intersected by drill core can be defined by two angles called alpha (α) and beta (β). The definition of these angles is shown in the diagram below:



4.8 Inclination

Term	Inclination (degrees from the horizontal)
Sub-horizontal	0-5
Gently Inclined	6-15
Moderately Inclined	16-30
Steeply Inclined	31-60
Very Steeply Inclined	61-80
Sub-vertical	81-90

4.9 Stratification/foliation

Term	Spacing
Very Thickly Bedded	>2m
Thickly Bedded	600mm-2m
Medium Bedded	200mm-600mm
Thinly Bedded	60mm-200mm

Term	Spacing
Very Thinly Bedded	20mm-60mm
Laminated	6mm-20mm
Thinly Laminated	2mm-6mm
Fissile	<2mm

4.10 Grain Size

Term	Size
Very Coarse Grained	>60 mm
Coarse Grained	2mm-60mm
Medium Grained	60 microns – 2mm
Fine Grained	2 microns – 60 microns
Very Fine Grained	<2 microns

4.11 Aperture of open discontinuity

Symbol	Aperture Opening	Description	
VT	<0.1 mm	Very tight	Closed Features
T	0.1-0.25mm	Tight	
PO	0.25-0.5mm	Partly open	
O	0.5-2.5mm	Open	Gapped Features
MW	2.5-10mm	Moderately open	
W	>10mm	Wide	
VW	1-10cm	Very wide	Open Features
EW	10-100cm	Extremely wide	
C	>1m	Cavernous	

4.12 Width of filled discontinuity

Symbol	Width	Description
W	12.5-50mm	Wide
MW	2.5-12.5mm	Moderately Wide
N	1.25-2.5mm	Narrow
VN	<1.25mm	Very Narrow
T	0mm	Tight

4.13 Roughness of discontinuity

Symbol	Description
Slk	Slickenside (surface has smooth, glassy finish with visual evidence of striations)
S	Smooth (surface appears smooth and feels so to the touch)
SR	Slightly rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
R	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)

Symbol	Description
VR	Very rough (near-vertical steps and ridges occur on the discontinuity surface)

4.14 Shape of discontinuity

Symbol	Description
Pl	Planar
St	Stepped
Un	Undulating
Ir	Irregular

4.15 Filling amount

Symbol	Description
Su	Surface Stain
Sp	Spotty
Pa	Partially Filled
Fi	Filled
No	None

4.16 Filling Type

Symbol	Term	Hard/Soft
Ab	Albite	Hard
Ah	Anhydrite	Hard
Bt	Biotite	Soft
Bn	Bornite	Hard
Ca	Calcite	Hard
Cb	Carbonate	Hard
Ch	Chlorite	Soft
Cpy	Chalcopyrite	Hard
Cy	Clay	Soft
Do	Dolomite	Hard
Ep	Epidote	Hard
Fd	Feldspar	Hard
FeOx	Iron Oxide	Hard
Go	Gouge	Soft
Gr	Graphite	Soft
Gy	Gypsum	Soft
He	Hematite	Hard
Ka	Kaolinite	Soft
Kf	K-feldspar	Hard

Symbol	Term	Hard/Soft
Lm	Limonite/FeOx	Soft
Ms	Muscovite	Soft
Mt	Magnetite	Hard
Py	Pyrite	Hard
Qz	Quartz	Hard
Rb	Rubble	Hard
Sa	Sand	Hard
Se	Sericite/Illite	Soft
Si	Silt	Hard
Sm	Smectite	Soft
Su	Sulphide	Hard
Ta	Talc	Soft
UH	Unknown Hard	Hard
US	Unknown Soft	Soft
OTH - see comments		

Appendix 4

Laboratory Results

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

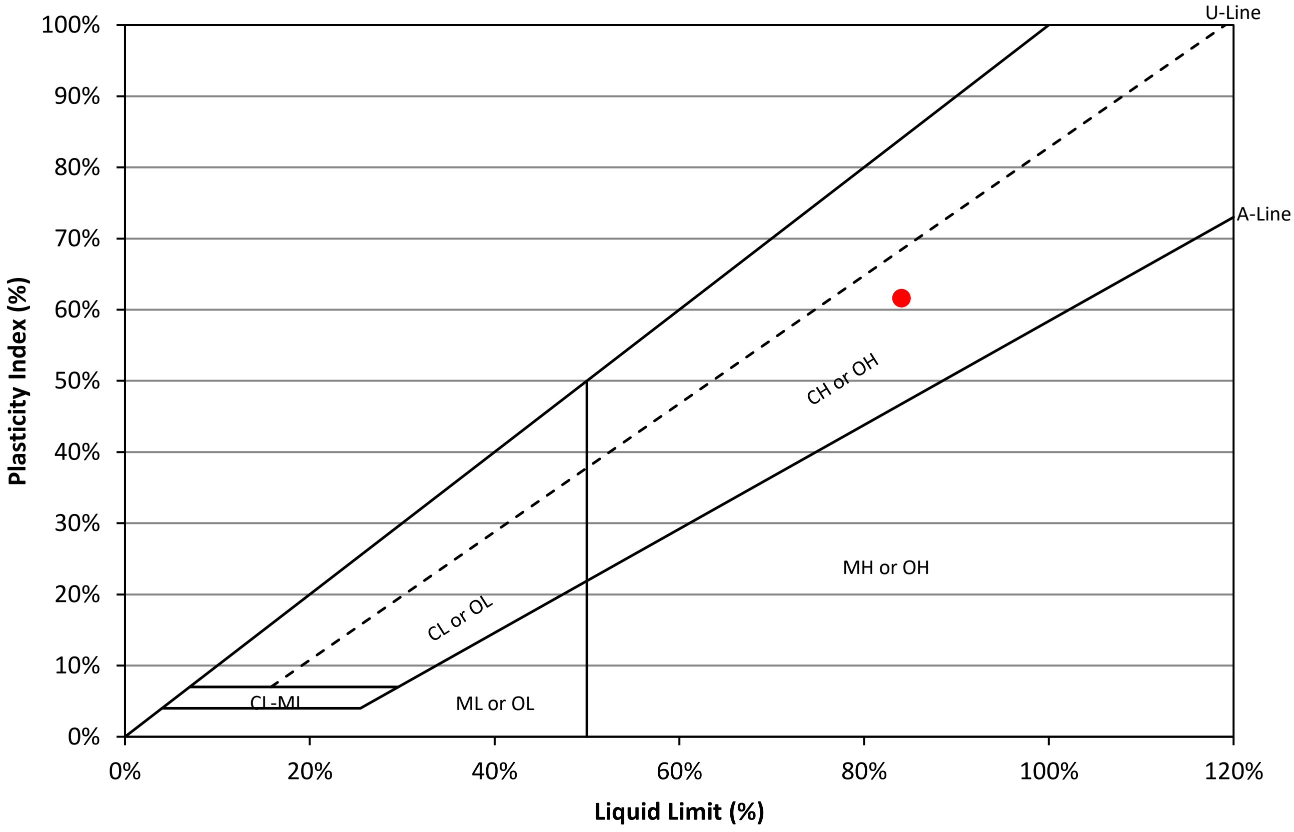
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	June 6, 2024
Sample Depth:	0.61 - 0.76 m	Lab Technician:	JEnriquez
Sample Number:	G2	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	27	19
Wet Sample (g)	13.0	12.6	13.0
Dry Sample (g)	7.3	6.9	6.9
Water Content (%)	78.0%	83.9%	89.9%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.4	6.2
Dry Sample (g)	5.2	5.1
Water Content (%)	22.3%	22.6%



Liquid Limit:	**84**	**Plastic Limit:**	**22**	**Plasticity Index:**	**62**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
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Phone: 204 477 5381

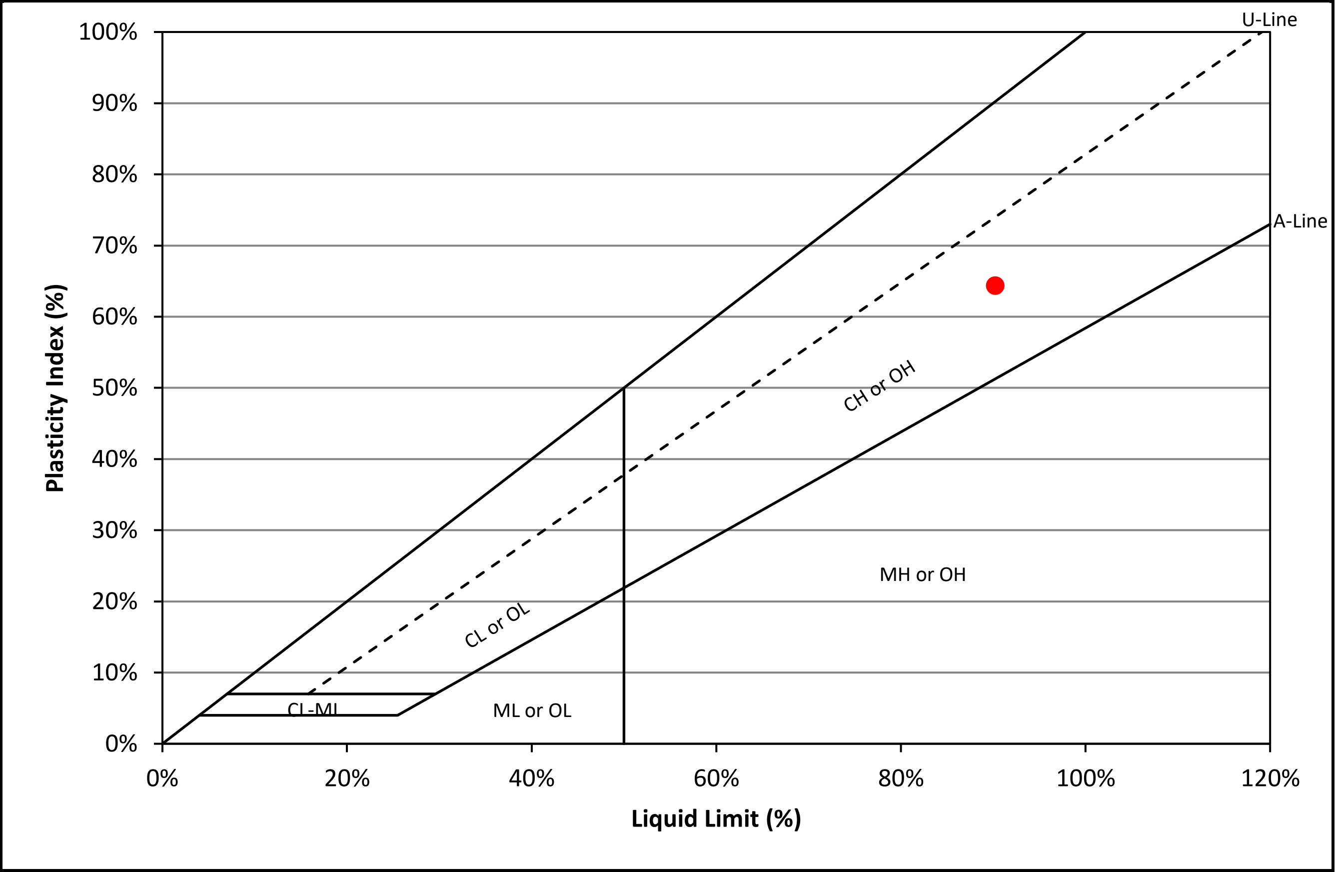
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	June 6, 2024
Sample Depth:	4.42 - 4.57 m	Lab Technician:	JEnriquez
Sample Number:	G6	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	24	20
Wet Sample (g)	12.1	12.0	12.4
Dry Sample (g)	6.6	6.3	6.4
Water Content (%)	83.8%	91.4%	94.6%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.8	7.5
Dry Sample (g)	5.4	6.0
Water Content (%)	26.2%	25.5%



Liquid Limit:	**90**	**Plastic Limit:**	**26**	**Plasticity Index:**	**64**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
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Phone: 204 477 5381

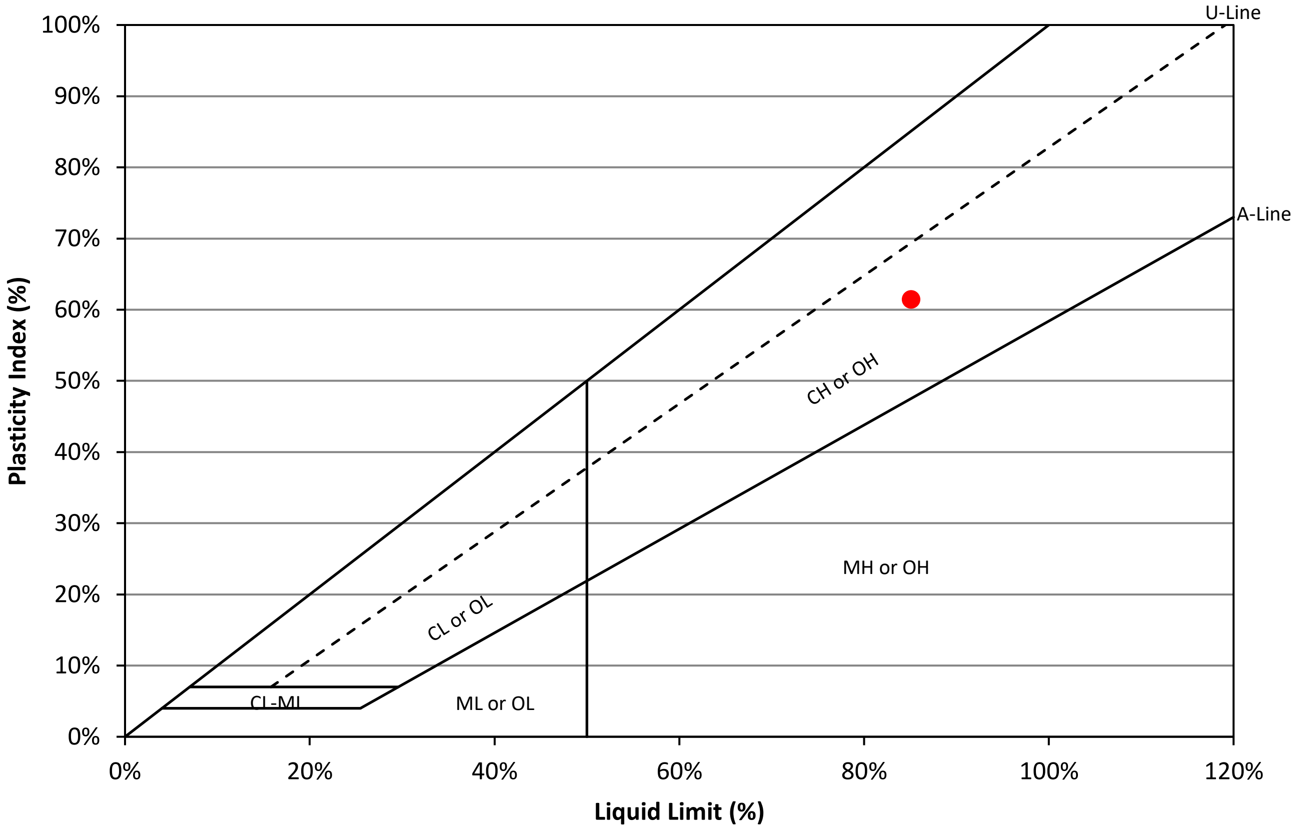
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	June 6, 2024
Sample Depth:	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	28	26	21
Wet Sample (g)	12.2	12.1	12.1
Dry Sample (g)	6.6	6.6	6.5
Water Content (%)	83.5%	84.4%	87.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.9	6.1
Dry Sample (g)	5.6	4.9
Water Content (%)	23.3%	24.1%



Liquid Limit:	**85**	**Plastic Limit:**	**24**	**Plasticity Index:**	**61**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Phone: 204 477 5381

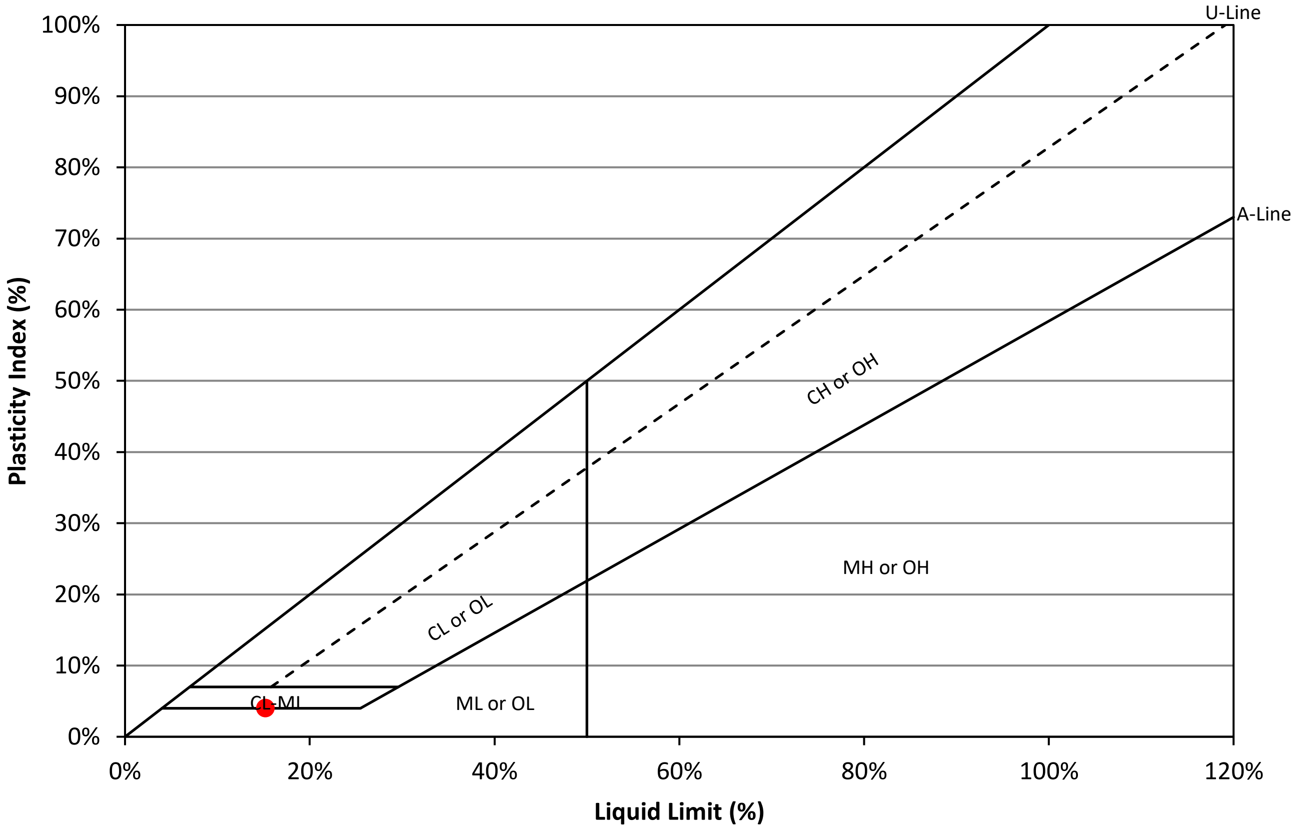
Project Name:	FGSV Siphon Replacement			
Project Number:	60728226		Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg		Field Technician:	GAcurin
Sample Location:	TH24-01		Sample Date:	June 6, 2024
Sample Depth:	16.61 - 16.76 m		Lab Technician:	JEnriquez
Sample Number:	G17		Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	15	22	28
Wet Sample (g)	14.4	13.1	12.9
Dry Sample (g)	12.2	11.3	11.3
Water Content (%)	18.5%	15.2%	15.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	9.7	9.5
Dry Sample (g)	8.7	8.5
Water Content (%)	10.7%	11.6%



Liquid Limit:	**15**	**Plastic Limit:**	**11**	**Plasticity Index:**	**4**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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Phone: 204 477 5381

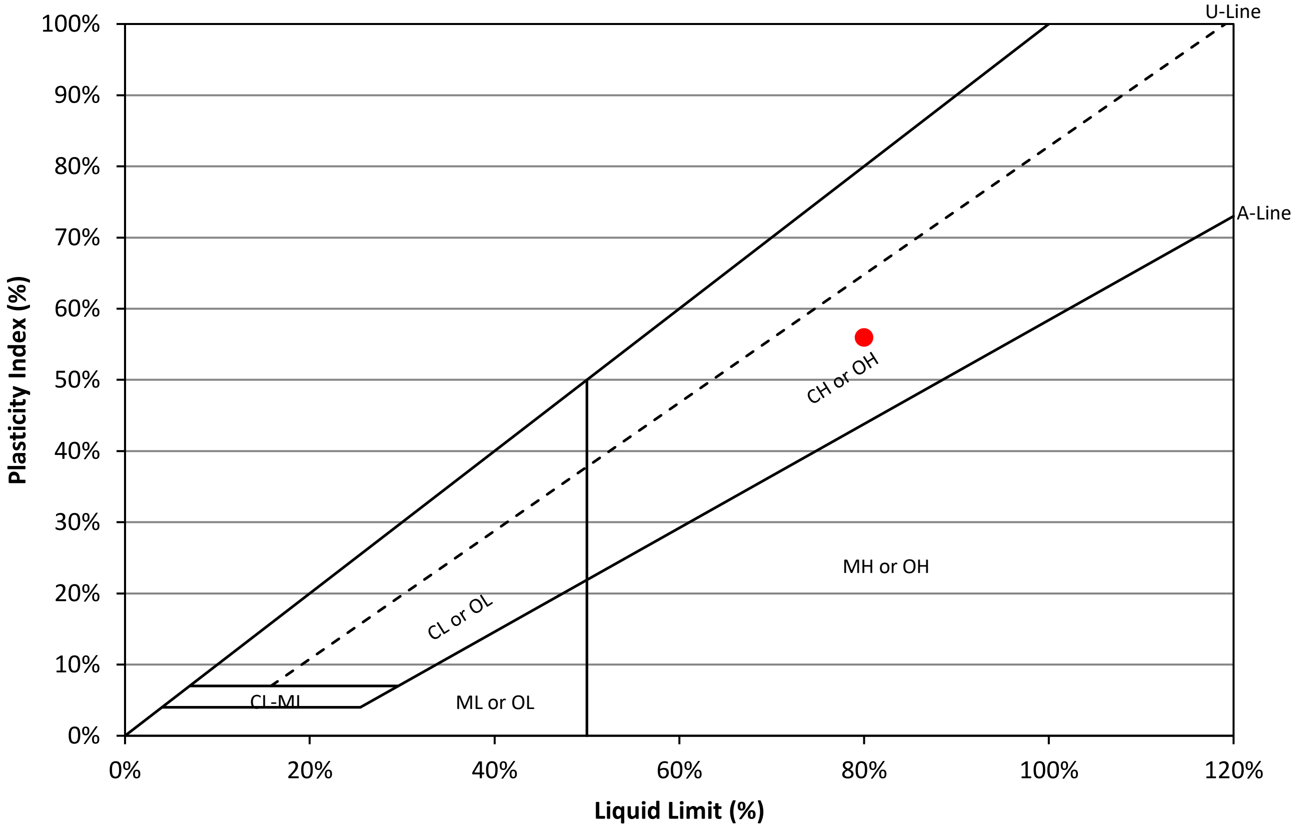
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	June 6, 2024
Sample Depth:	5.94 - 6.10 m	Lab Technician:	JEnriquez
Sample Number:	G7	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	29	20	15
Wet Sample (g)	13.5	12.5	12.4
Dry Sample (g)	7.6	6.8	6.7
Water Content (%)	77.7%	84.0%	85.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.9	8.7
Dry Sample (g)	5.6	7.0
Water Content (%)	23.2%	25.0%



Liquid Limit:	**80**	**Plastic Limit:**	**24**	**Plasticity Index:**	**56**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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Phone: 204 477 5381

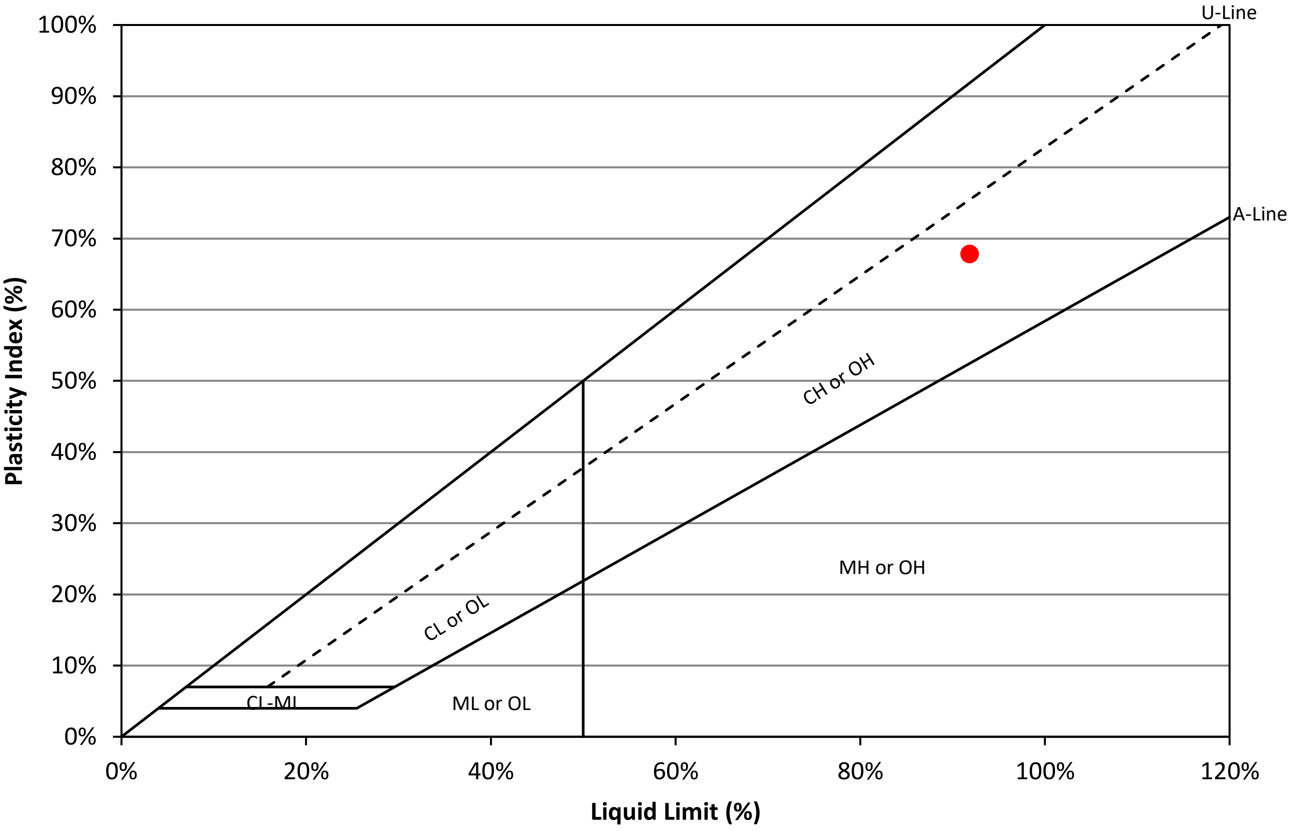
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	June 6, 2024
Sample Depth:	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	26	23	15
Wet Sample (g)	12.7	10.9	12.4
Dry Sample (g)	6.6	5.6	6.3
Water Content (%)	91.3%	92.9%	96.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.8	6.4
Dry Sample (g)	5.5	5.2
Water Content (%)	23.8%	24.3%



Liquid Limit:	**92**	**Plastic Limit:**	**24**	**Plasticity Index:**	**68**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
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Phone: 204 477 5381

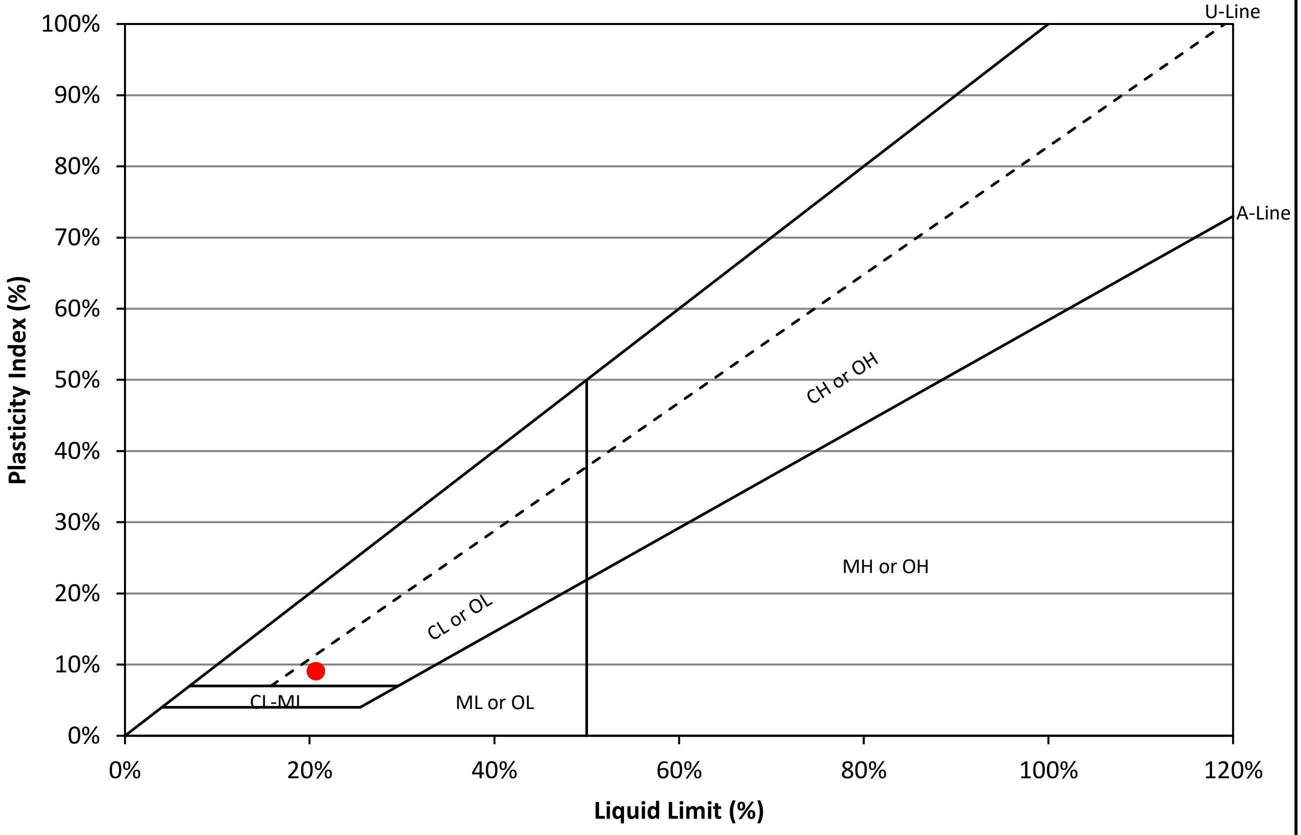
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	June 6, 2024
Sample Depth:	12.04 - 12.19 m	Lab Technician:	JEnriquez
Sample Number:	G13	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	25	23
Wet Sample (g)	13.4	11.4	14.5
Dry Sample (g)	11.2	9.5	11.9
Water Content (%)	19.1%	20.5%	21.4%

Plastic Limit		
Trial	1	2
Wet Sample (g)	14.4	14.0
Dry Sample (g)	12.8	12.6
Water Content (%)	12.0%	11.2%



Liquid Limit:	**21**	**Plastic Limit:**	**12**	**Plasticity Index:**	**9**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Winnipeg Geotechnical Laboratory
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Phone: 204 477 5381

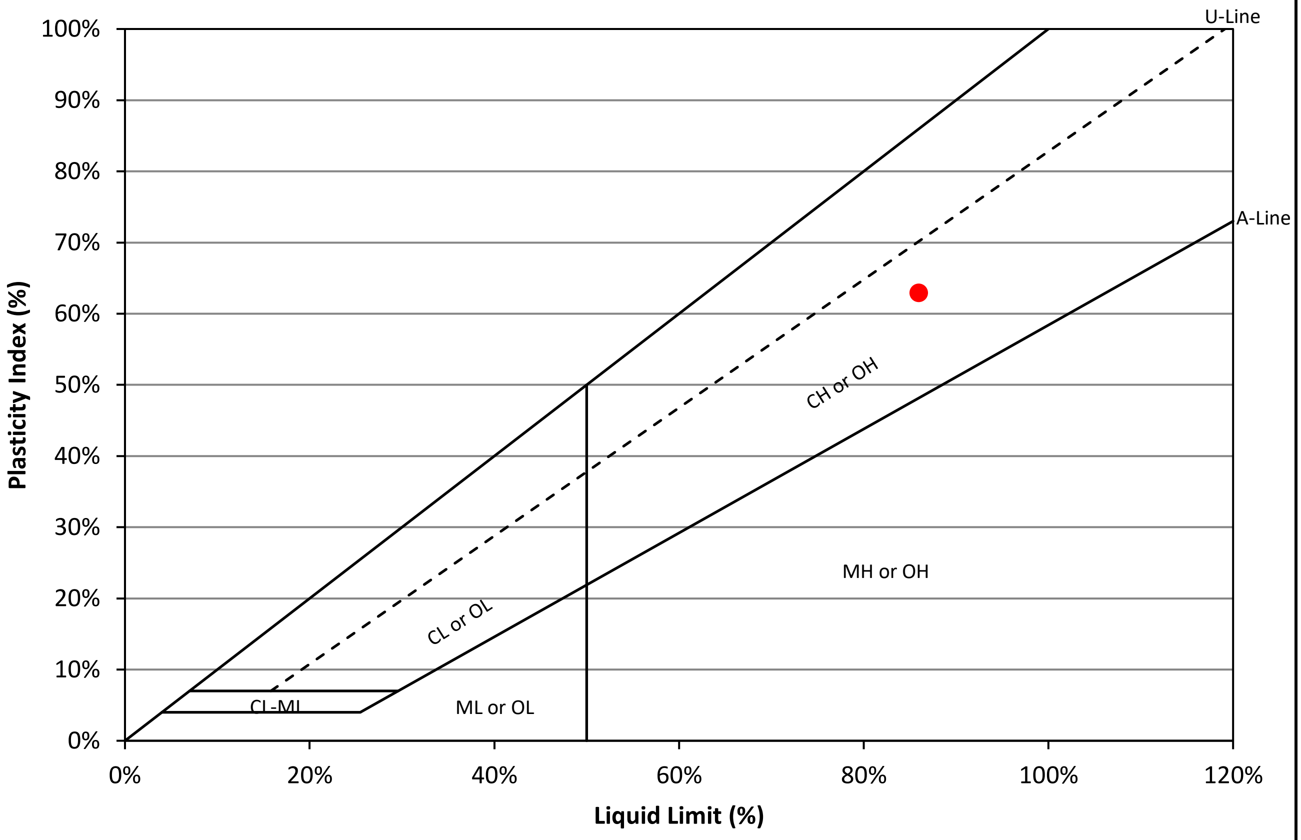
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	June 6, 2024
Sample Depth:	5.94 - 6.10 m	Lab Technician:	JEnriquez
Sample Number:	G7	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	27	23	16
Wet Sample (g)	12.2	10.8	11.0
Dry Sample (g)	6.6	5.7	5.8
Water Content (%)	84.1%	88.5%	88.9%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.3	7.3
Dry Sample (g)	5.2	5.9
Water Content (%)	22.5%	23.7%



Liquid Limit:	**86**	**Plastic Limit:**	**23**	**Plasticity Index:**	**63**

Reviewed by: Lee Boughton
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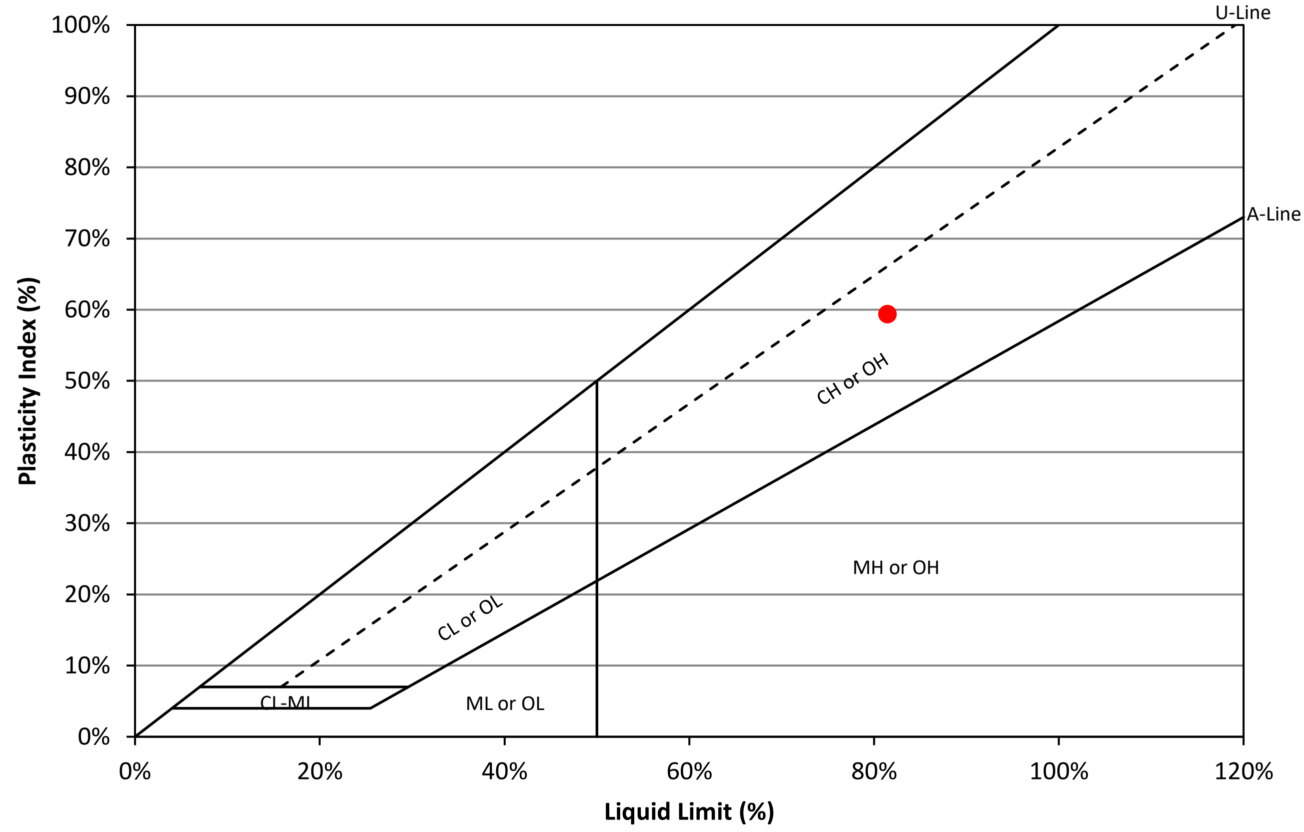
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	June 6, 2024
Sample Depth:	8.99 - 9.14 m	Lab Technician:	JEnriquez
Sample Number:	G10	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	24	19
Wet Sample (g)	12.3	11.1	12.5
Dry Sample (g)	6.9	6.1	6.7
Water Content (%)	79.2%	81.6%	84.8%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.7	7.0
Dry Sample (g)	5.5	5.8
Water Content (%)	22.8%	21.4%



Liquid Limit:	**81**	**Plastic Limit:**	**22**	**Plasticity Index:**	**59**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name: FGSV Siphon Replacement

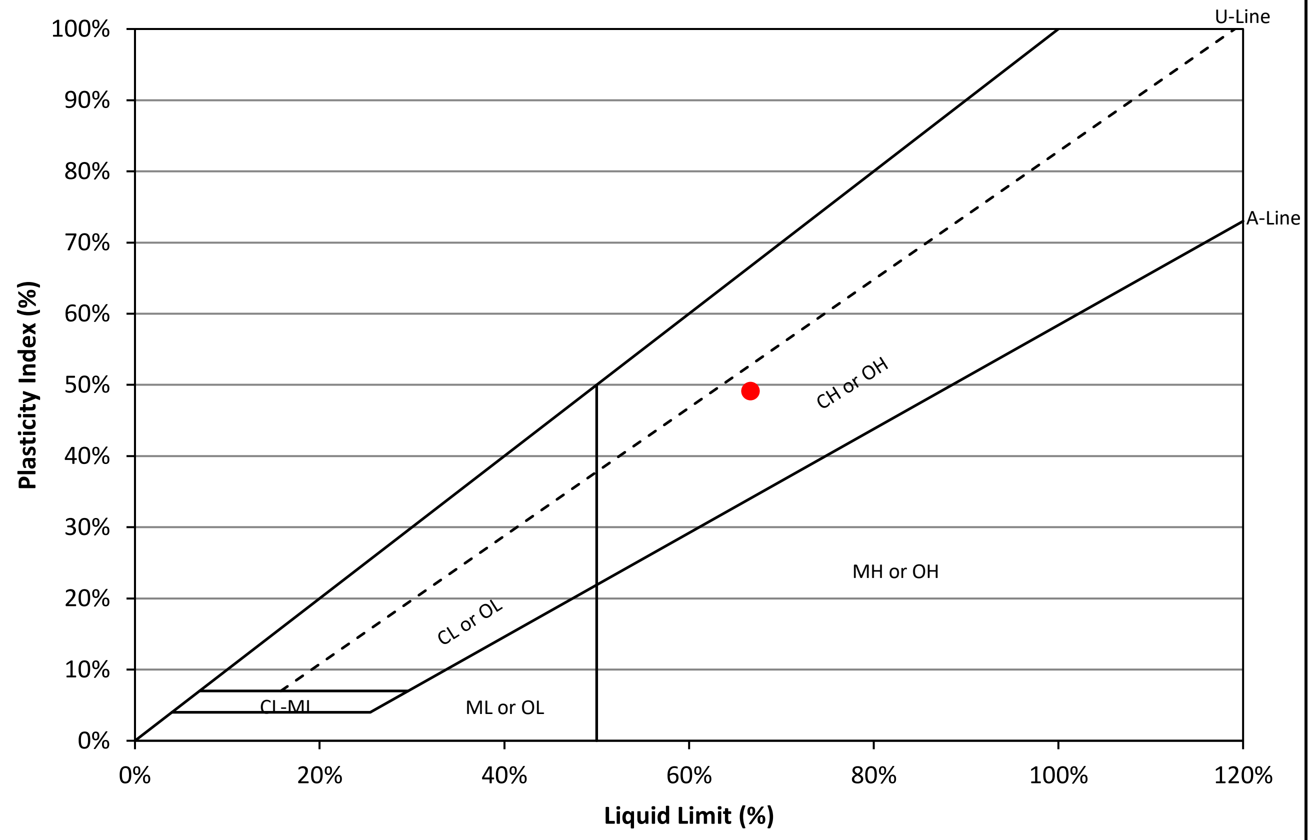
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	June 6, 2024
Sample Depth:	12.04 - 12.19 m	Lab Technician:	JEnriquez
Sample Number:	G13	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	27	16
Wet Sample (g)	11.0	14.3	14.1
Dry Sample (g)	6.7	8.6	8.3
Water Content (%)	63.6%	66.7%	70.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.2	8.1
Dry Sample (g)	6.1	6.9
Water Content (%)	17.4%	17.7%



Liquid Limit:	**67**	**Plastic Limit:**	**18**	**Plasticity Index:**	**49**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

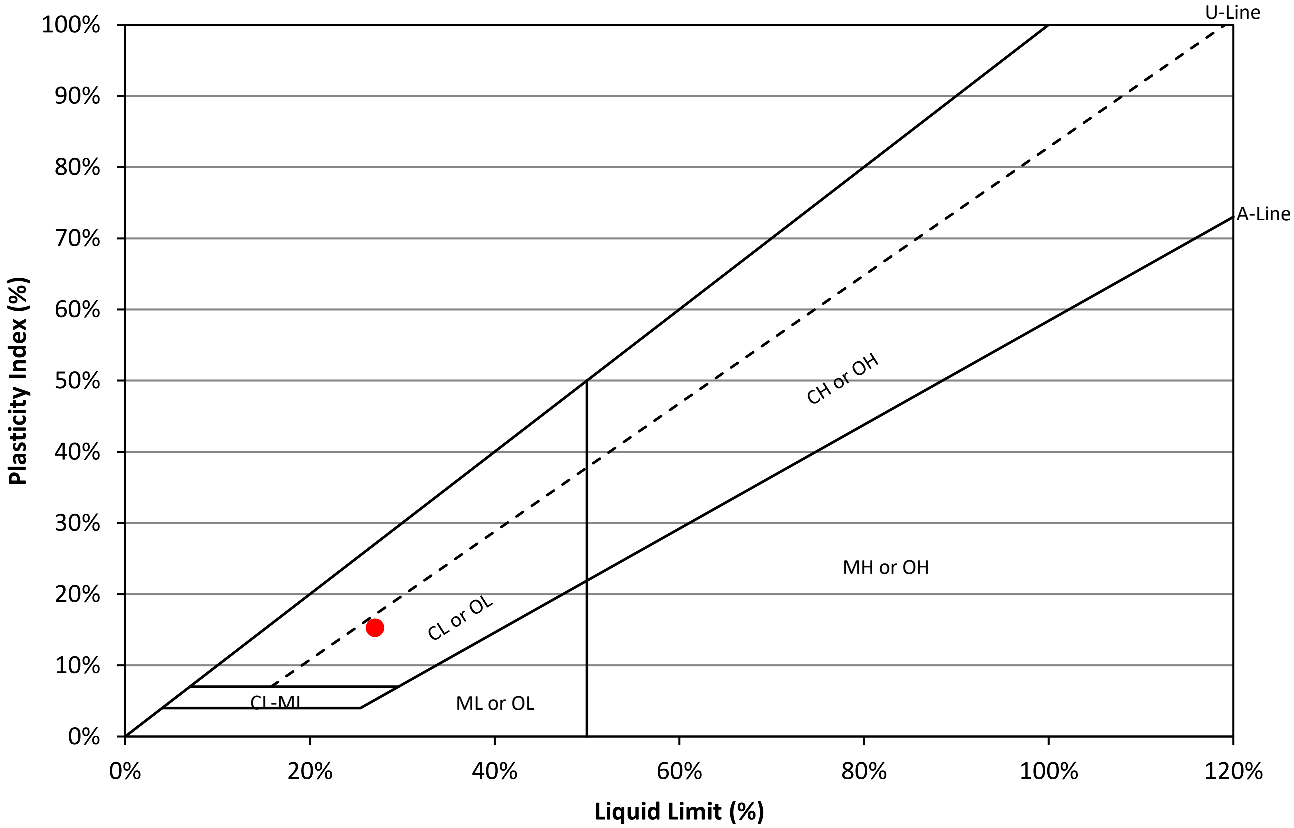
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	June 6, 2024
Sample Depth:	12.95 - 13.11 m	Lab Technician:	JEnriquez
Sample Number:	G14	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	27	23
Wet Sample (g)	12.6	13.5	12.3
Dry Sample (g)	10.0	10.7	9.6
Water Content (%)	25.0%	26.2%	27.9%

Plastic Limit		
Trial	1	2
Wet Sample (g)	9.0	7.8
Dry Sample (g)	8.1	7.0
Water Content (%)	11.7%	11.9%



Liquid Limit:	**27**	**Plastic Limit:**	**12**	**Plasticity Index:**	**15**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

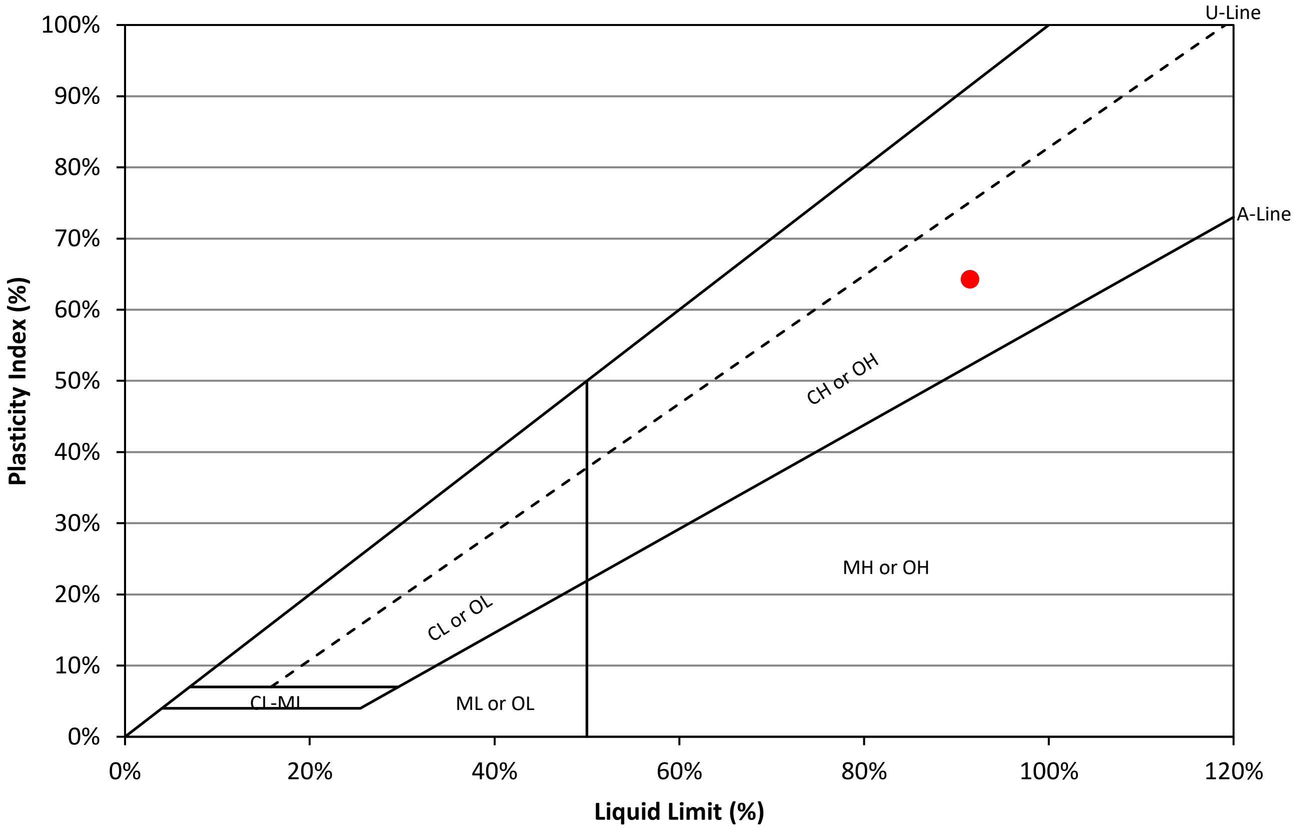
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	June 6, 2024
Sample Depth:	0.76 - 0.91 m	Lab Technician:	JEnriquez
Sample Number:	G2	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	26	20	18
Wet Sample (g)	14.2	11.1	11.5
Dry Sample (g)	7.4	5.7	5.9
Water Content (%)	90.8%	95.3%	96.1%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.5	6.3
Dry Sample (g)	5.1	4.9
Water Content (%)	27.0%	27.4%



Liquid Limit:	**91**	**Plastic Limit:**	**27**	**Plasticity Index:**	**64**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Phone: 204 477 5381

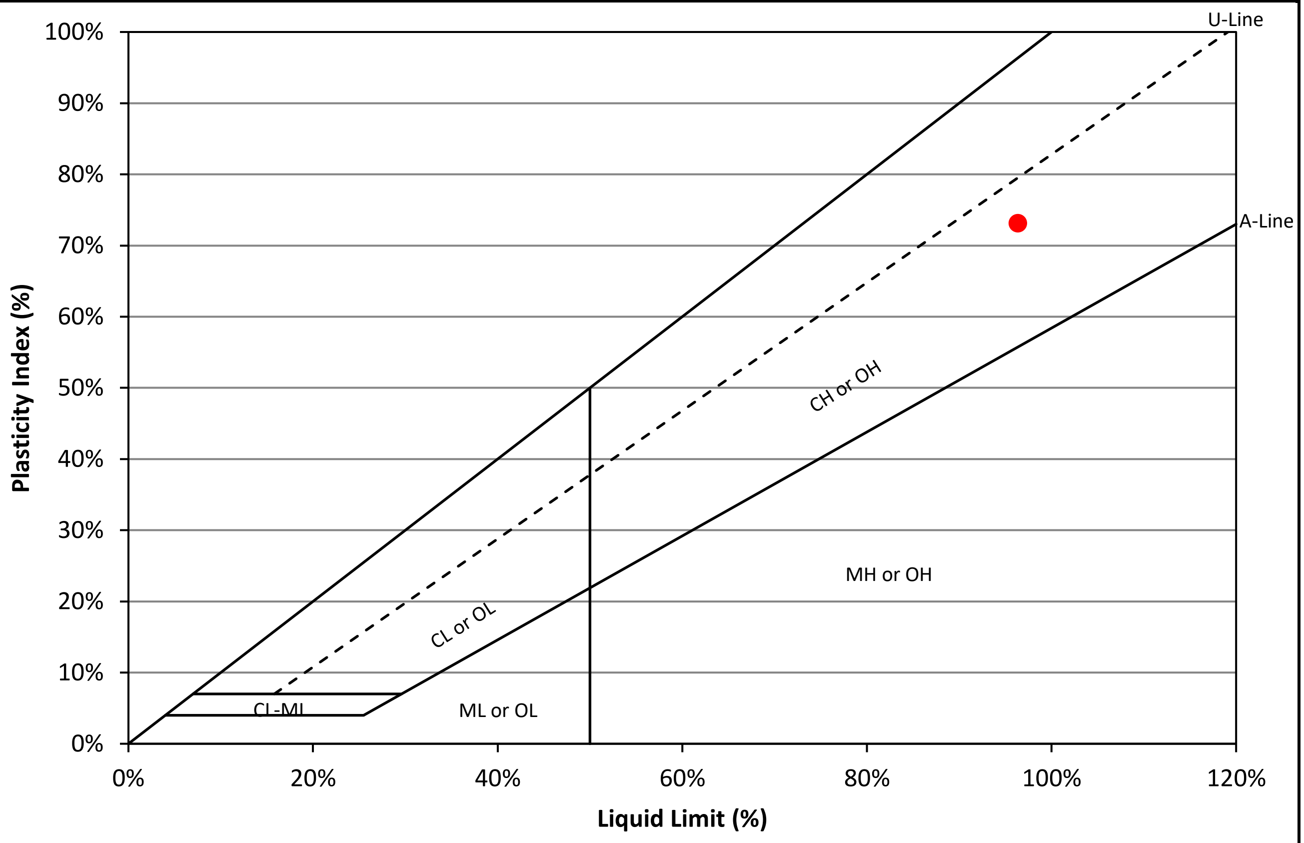
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	June 6, 2024
Sample Depth:	4.42 - 4.57 m	Lab Technician:	JEnriquez
Sample Number:	G6	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	31	29	18
Wet Sample (g)	10.7	12.0	13.2
Dry Sample (g)	5.6	6.2	6.5
Water Content (%)	92.5%	94.3%	101.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.6	6.7
Dry Sample (g)	6.1	5.4
Water Content (%)	23.3%	23.1%



Liquid Limit:	**96**	**Plastic Limit:**	**23**	**Plasticity Index:**	**73**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
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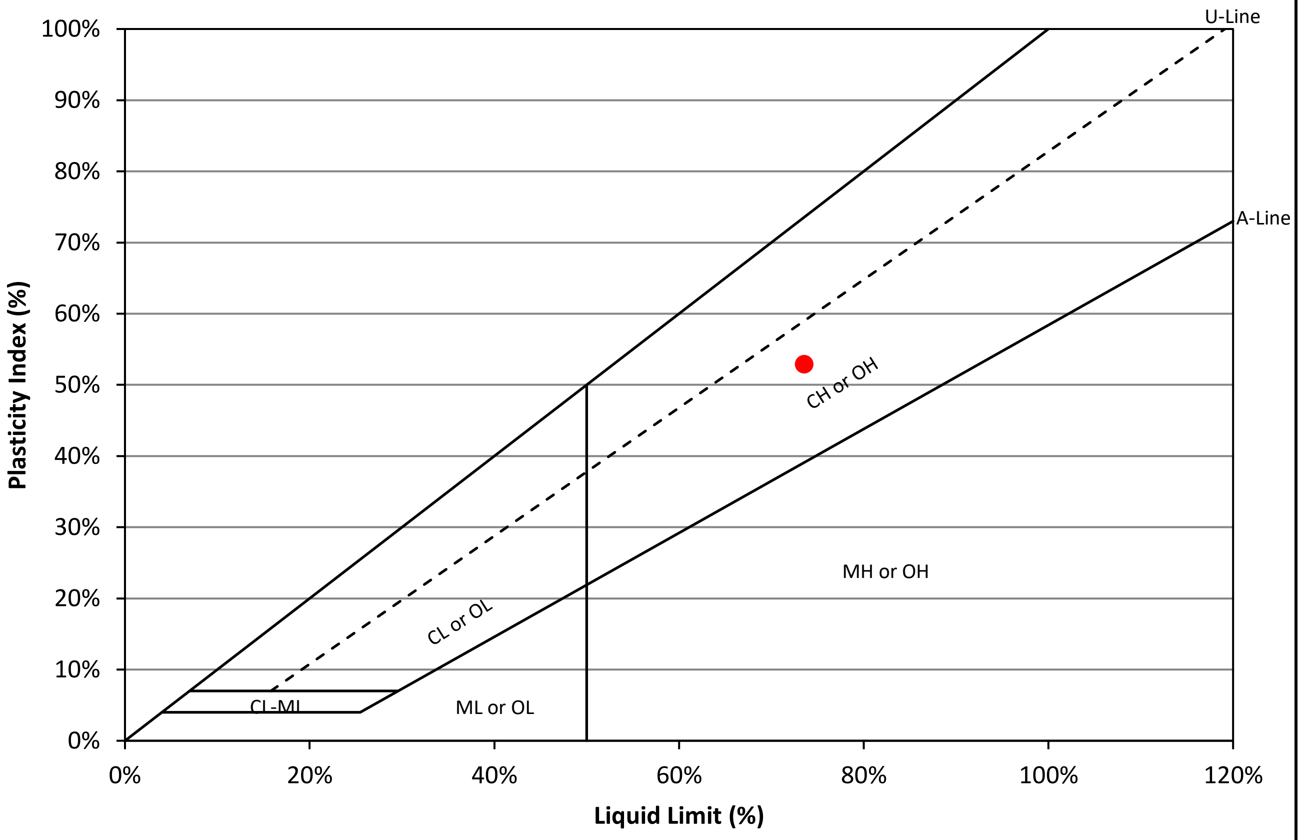
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	June 6, 2024
Sample Depth:	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	25	20	15
Wet Sample (g)	13.8	12.3	14.5
Dry Sample (g)	8.0	6.9	8.1
Water Content (%)	73.1%	77.2%	79.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.6	7.0
Dry Sample (g)	5.5	5.8
Water Content (%)	21.0%	20.3%



Liquid Limit:	**74**	**Plastic Limit:**	**21**	**Plasticity Index:**	**53**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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Project Name: FGSV Siphon Replacement

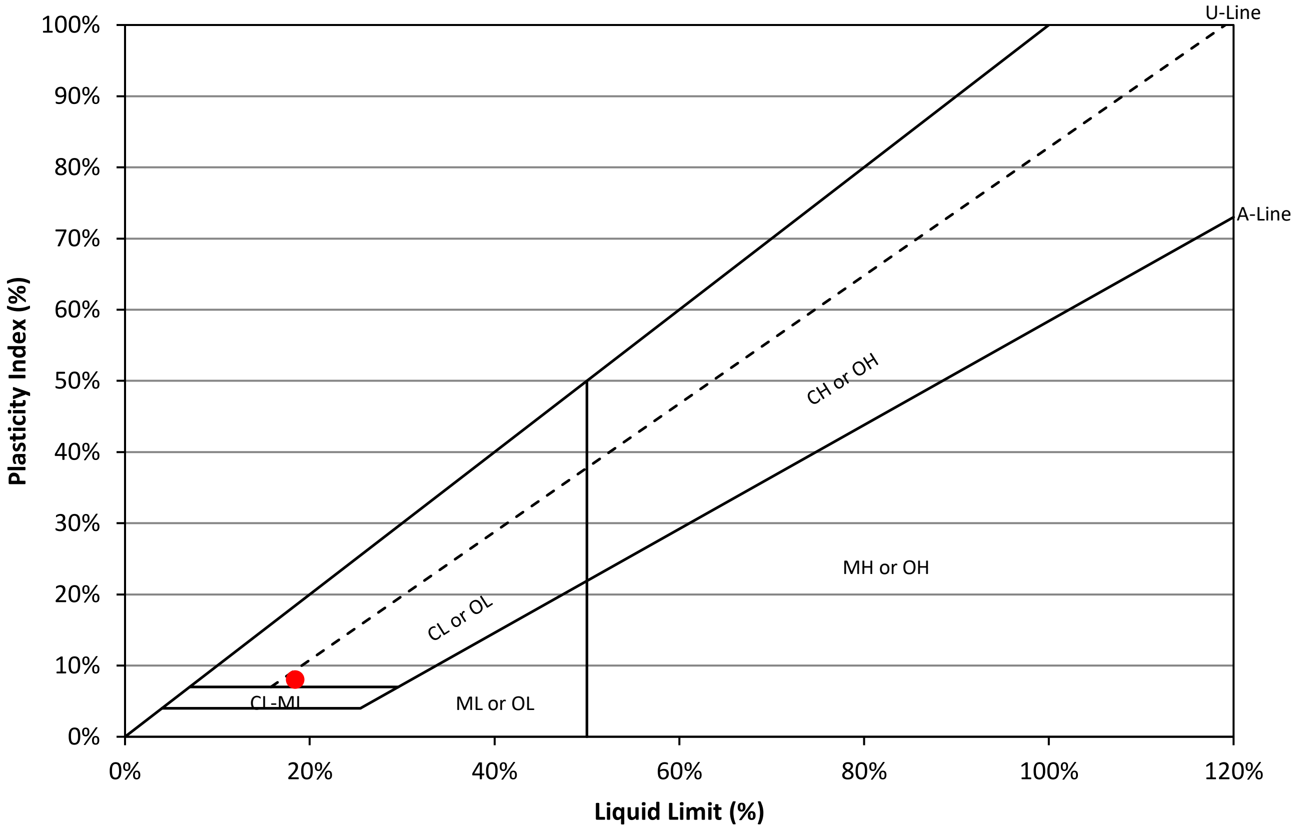
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	June 6, 2024
Sample Depth:	13.56 - 13.72 m	Lab Technician:	JEnriquez
Sample Number:	G15	Date Tested:	June 18, 2024

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	18	24	26
Wet Sample (g)	13.1	14.3	13.8
Dry Sample (g)	11.0	12.0	11.7
Water Content (%)	19.2%	18.6%	18.2%

Plastic Limit		
Trial	1	2
Wet Sample (g)	8.3	8.3
Dry Sample (g)	7.5	7.5
Water Content (%)	10.5%	10.4%



Liquid Limit:	**18**	**Plastic Limit:**	**10**	**Plasticity Index:**	**8**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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AECOM Canada Ltd.
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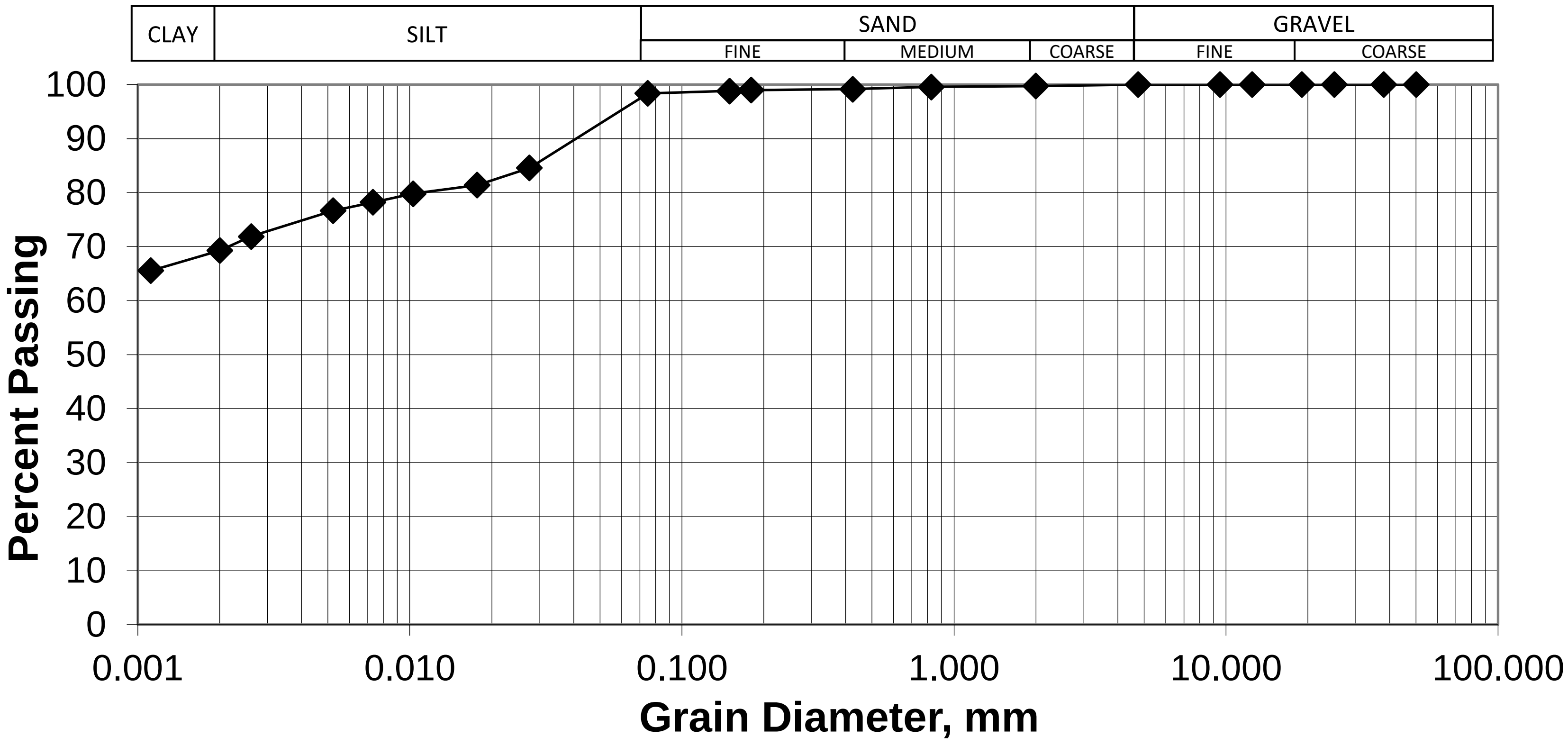
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	6-Jun-24
Sample Depth :	0.61 - 0.76 m	Lab Technician:	JEnriquez
Sample Number:	G2	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	98.4
38.0	100.0	2.00	99.7	0.0275	84.8
25.0	100.0	0.825	99.6	0.0177	81.6
19.0	100.0	0.425	99.1	0.0103	80.0
12.5	100.0	0.18	99.0	0.0073	78.4
9.5	100.0	0.15	98.8	0.0052	76.8
4.75	100.0	0.075	98.4	0.0026	72.1
				0.0020	69.5
				0.0011	65.7

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**28.9%**
Sand	**1.6%**	**Clay**	**69.5%**

Reviewed by:	Lee Boughton Laboratory Manager	Approved by:	German Leal, M.Eng., P.Eng. Geotechnical Discipline Lead

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Phone: 204 477 5381

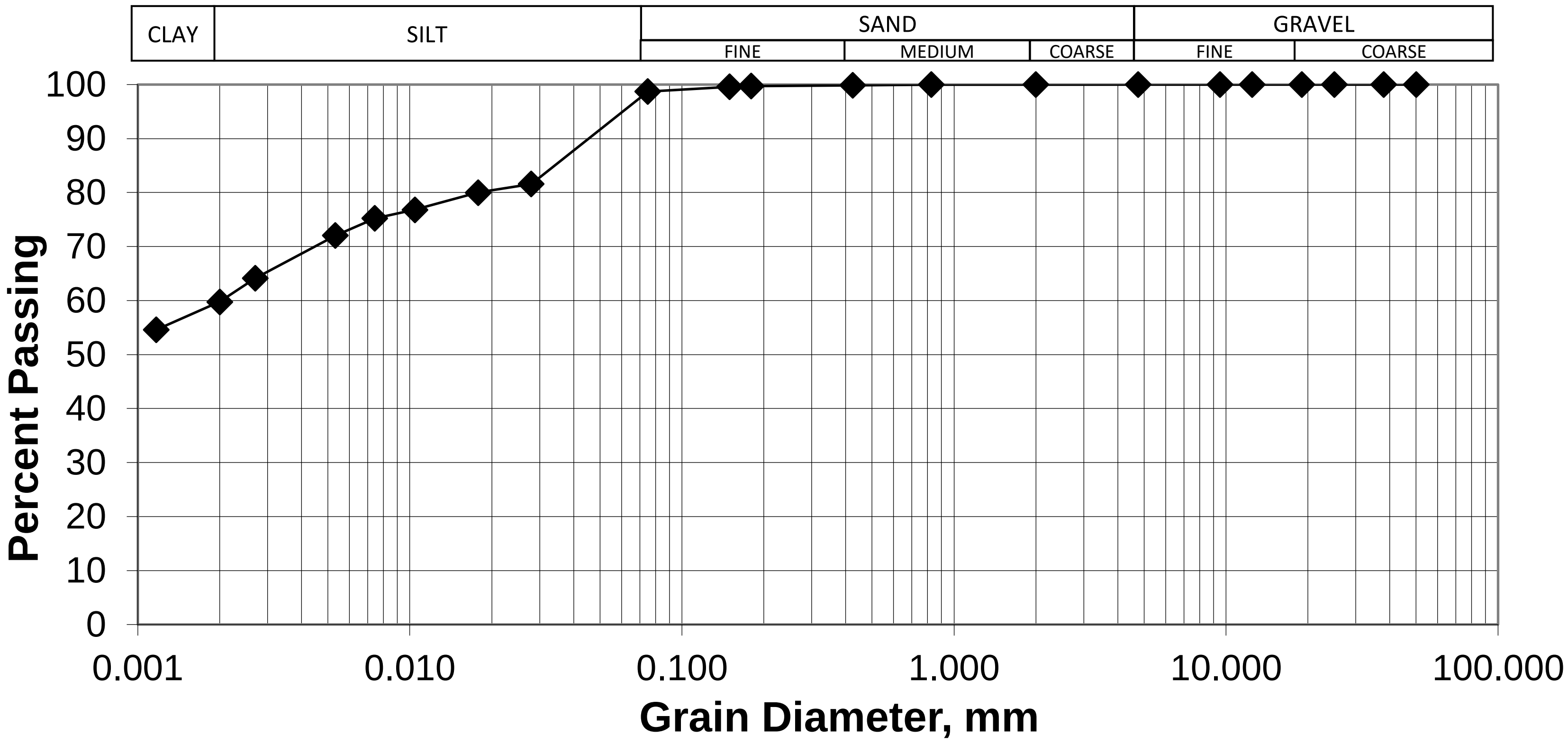
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	6-Jun-24
Sample Depth :	4.42 - 4.57 m	Lab Technician:	JEnriquez
Sample Number:	G6	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	98.7
38.0	100.0	2.00	100.0	0.0280	81.6
25.0	100.0	0.825	100.0	0.0178	80.0
19.0	100.0	0.425	99.9	0.0104	76.8
12.5	100.0	0.18	99.7	0.0074	75.2
9.5	100.0	0.15	99.6	0.0053	72.1
4.75	100.0	0.075	98.7	0.0027	64.1
				0.0020	59.8
				0.0012	54.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**38.9%**
Sand	**1.3%**	**Clay**	**59.8%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
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Phone: 204 477 5381

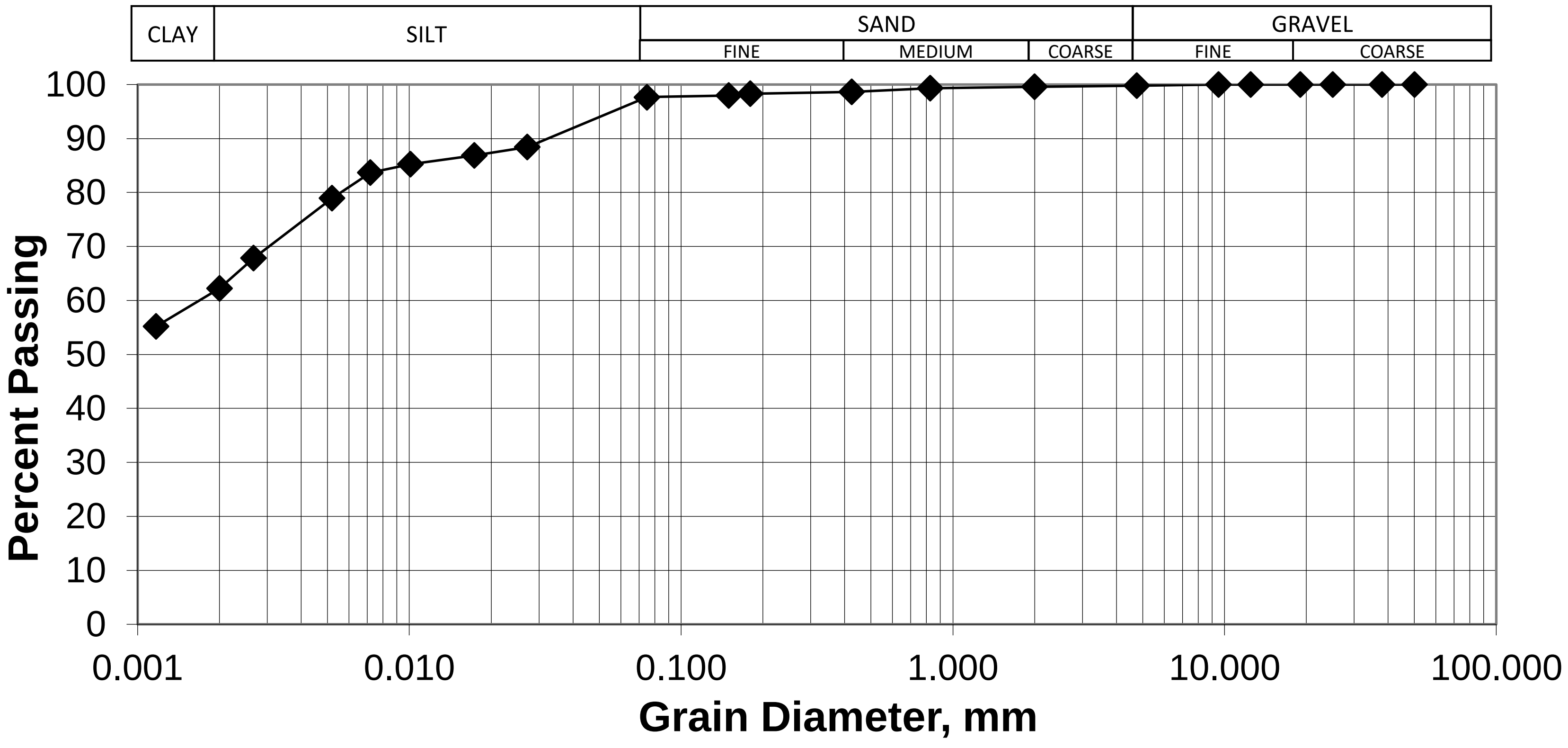
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	6-Jun-24
Sample Depth :	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	99.8	0.0750	97.7
38.0	100.0	2.00	99.6	0.0272	88.8
25.0	100.0	0.825	99.3	0.0173	87.2
19.0	100.0	0.425	98.7	0.0101	85.6
12.5	100.0	0.18	98.3	0.0072	84.0
9.5	100.0	0.15	98.0	0.0052	79.3
4.75	99.8	0.075	97.7	0.0027	68.2
				0.0020	62.5
				0.0012	55.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.2%**	**Silt**	**35.2%**
Sand	**2.2%**	**Clay**	**62.5%**

Reviewed by:	Lee Boughton Laboratory Manager	Approved by:	German Leal, M.Eng., P.Eng. Geotechnical Discipline Lead

AECOM Canada Ltd.
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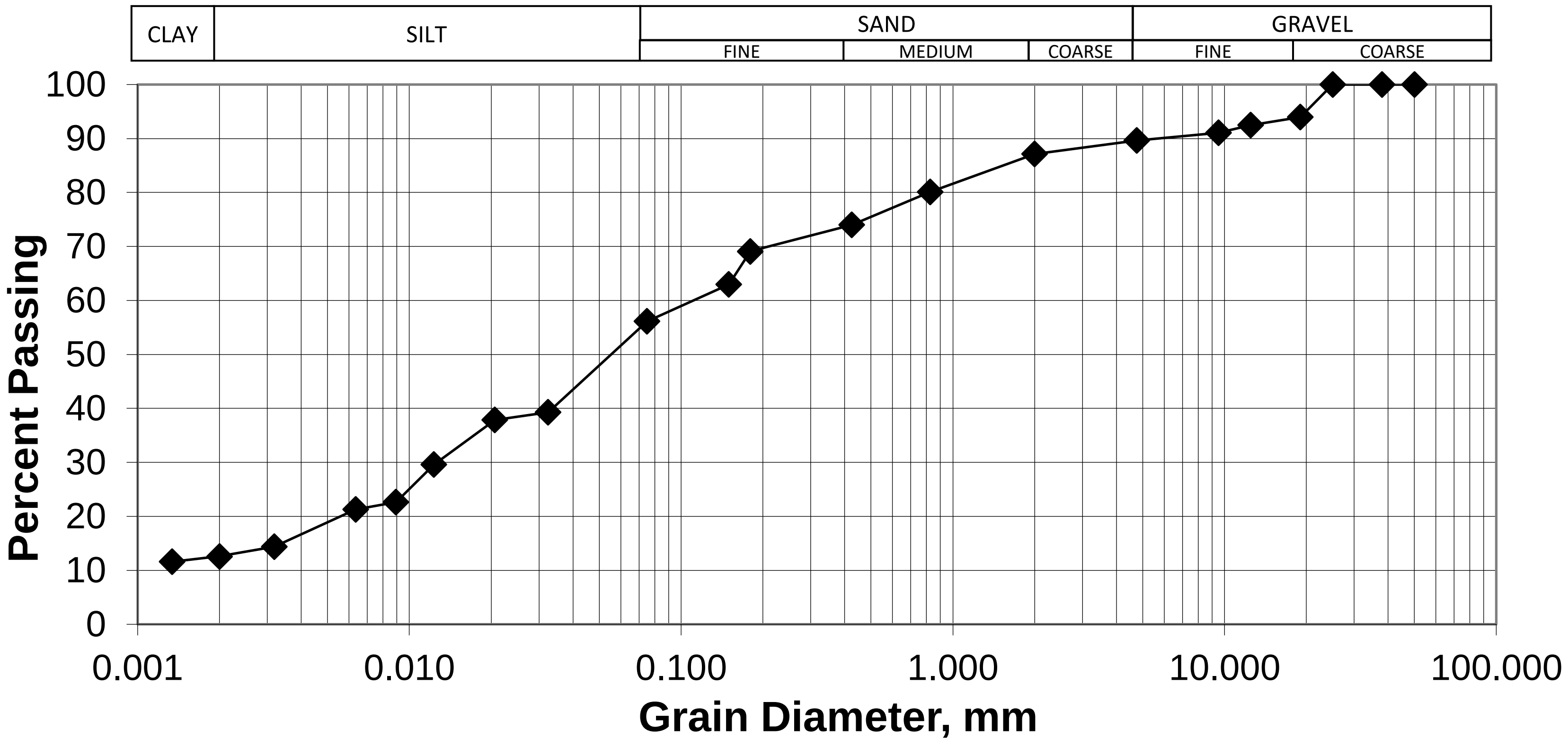
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-01	Sample Date:	6-Jun-24
Sample Depth :	12.04 - 12.19 m	Lab Technician:	JEnriquez
Sample Number:	G17	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	89.6	0.0750	56.2
38.0	100.0	2.00	87.1	0.0325	45.1
25.0	100.0	0.825	80.1	0.0206	43.5
19.0	93.9	0.425	74.0	0.0123	33.9
12.5	92.5	0.18	69.1	0.0089	26.0
9.5	91.1	0.15	63.0	0.0063	24.4
4.75	89.6	0.075	56.2	0.0032	16.5
				0.0020	14.4
				0.0013	13.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**10.4%**	**Silt**	**41.7%**
Sand	**33.5%**	**Clay**	**14.4%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Phone: 204 477 5381

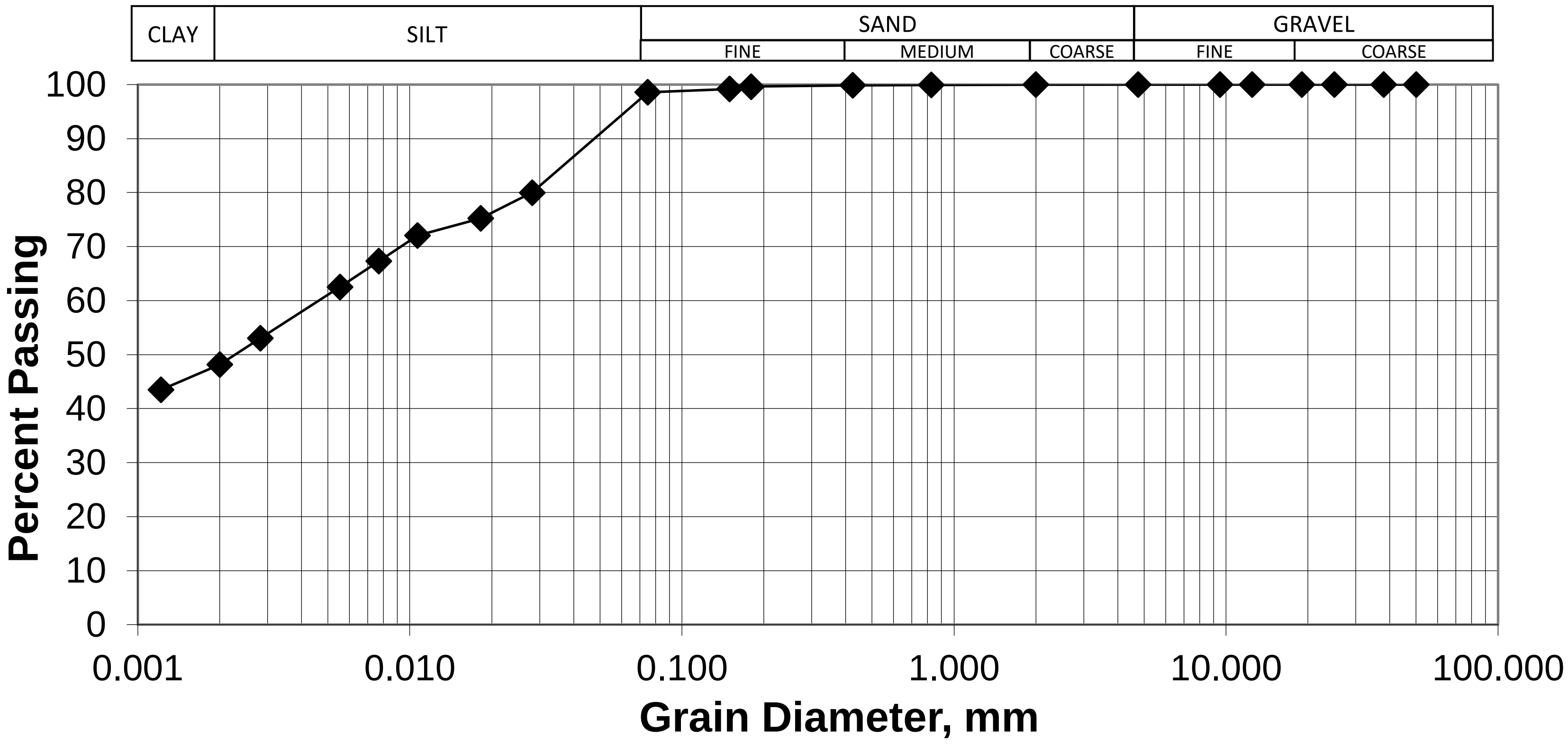
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	6-Jun-24
Sample Depth :	5.94 - 6.10 m	Lab Technician:	JEnriquez
Sample Number:	G7	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	98.6
38.0	100.0	2.00	100.0	0.0282	80.0
25.0	100.0	0.825	100.0	0.0182	75.2
19.0	100.0	0.425	99.9	0.0107	72.1
12.5	100.0	0.18	99.6	0.0077	67.3
9.5	100.0	0.15	99.2	0.0055	62.5
4.75	100.0	0.075	98.6	0.0028	53.0
				0.0020	48.1
				0.0012	43.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**50.4%**
Sand	**1.4%**	**Clay**	**48.1%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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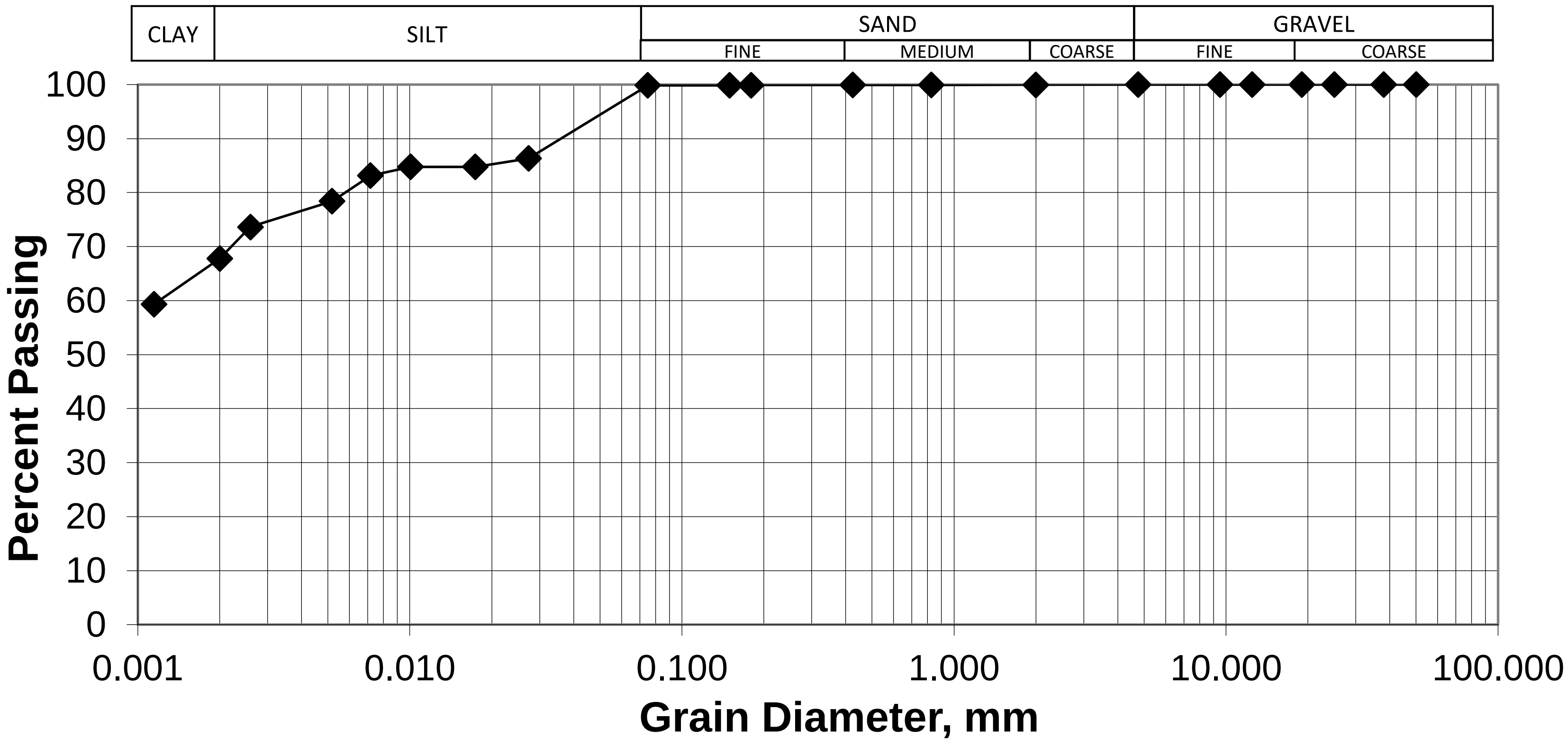
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	6-Jun-24
Sample Depth :	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	99.8
38.0	100.0	2.00	100.0	0.0273	86.4
25.0	100.0	0.825	99.9	0.0174	84.8
19.0	100.0	0.425	99.9	0.0101	84.8
12.5	100.0	0.18	99.9	0.0072	83.2
9.5	100.0	0.15	99.9	0.0052	78.4
4.75	100.0	0.075	99.8	0.0026	73.7
				0.0020	67.8
				0.0011	59.4

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**32.1%**
Sand	**0.2%**	**Clay**	**67.8%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
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Phone: 204 477 5381

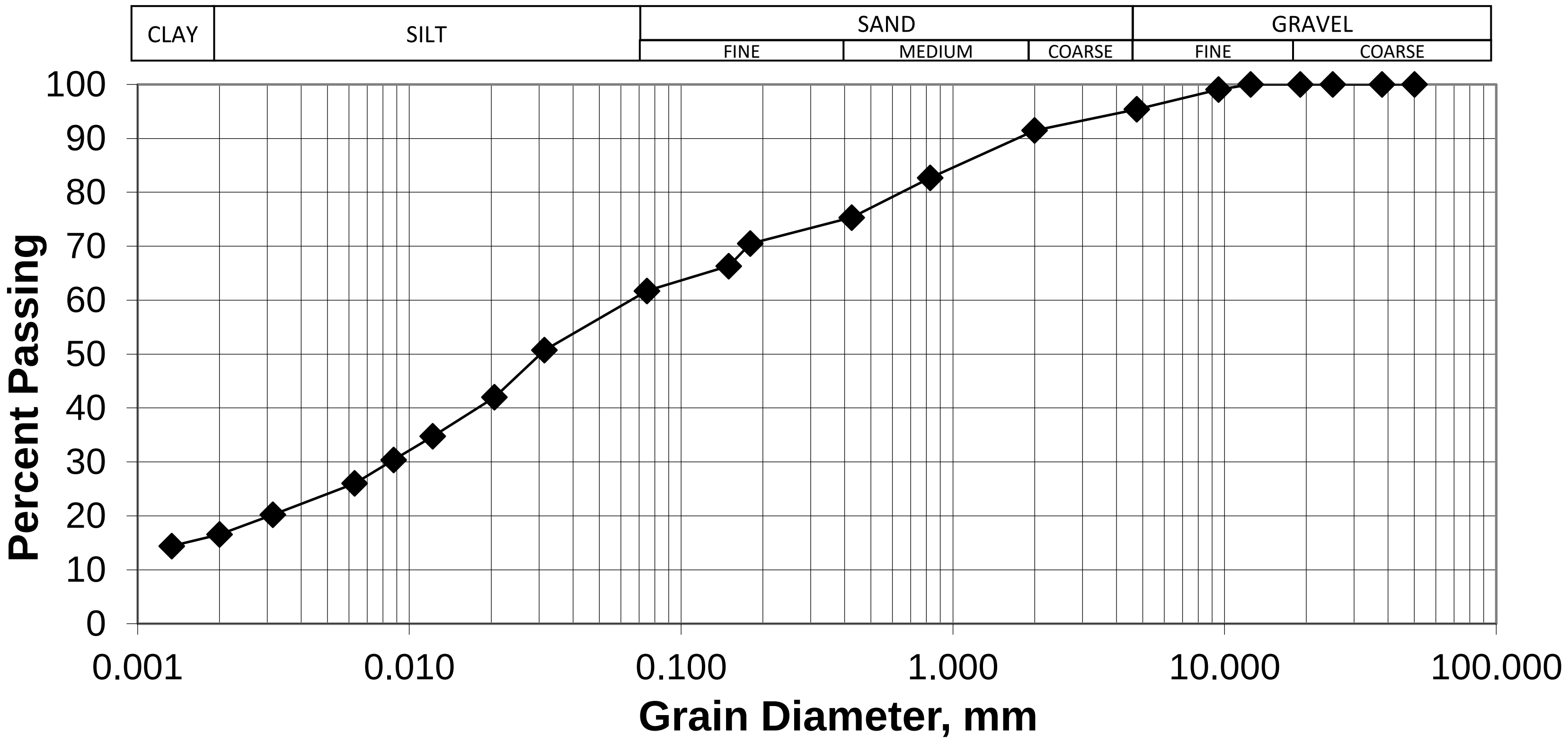
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-02	Sample Date:	6-Jun-24
Sample Depth :	12.04 - 12.19 m	Lab Technician:	JEnriquez
Sample Number:	G13	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	95.4	0.0750	61.7
38.0	100.0	2.00	91.4	0.0314	55.5
25.0	100.0	0.825	82.7	0.0206	45.9
19.0	100.0	0.425	75.3	0.0122	38.0
12.5	100.0	0.18	70.5	0.0088	33.2
9.5	99.0	0.15	66.3	0.0063	28.5
4.75	95.4	0.075	61.7	0.0031	22.1
				0.0020	18.1
				0.0013	15.8

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**4.6%**	**Silt**	**43.6%**
Sand	**33.6%**	**Clay**	**18.1%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
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Phone: 204 477 5381

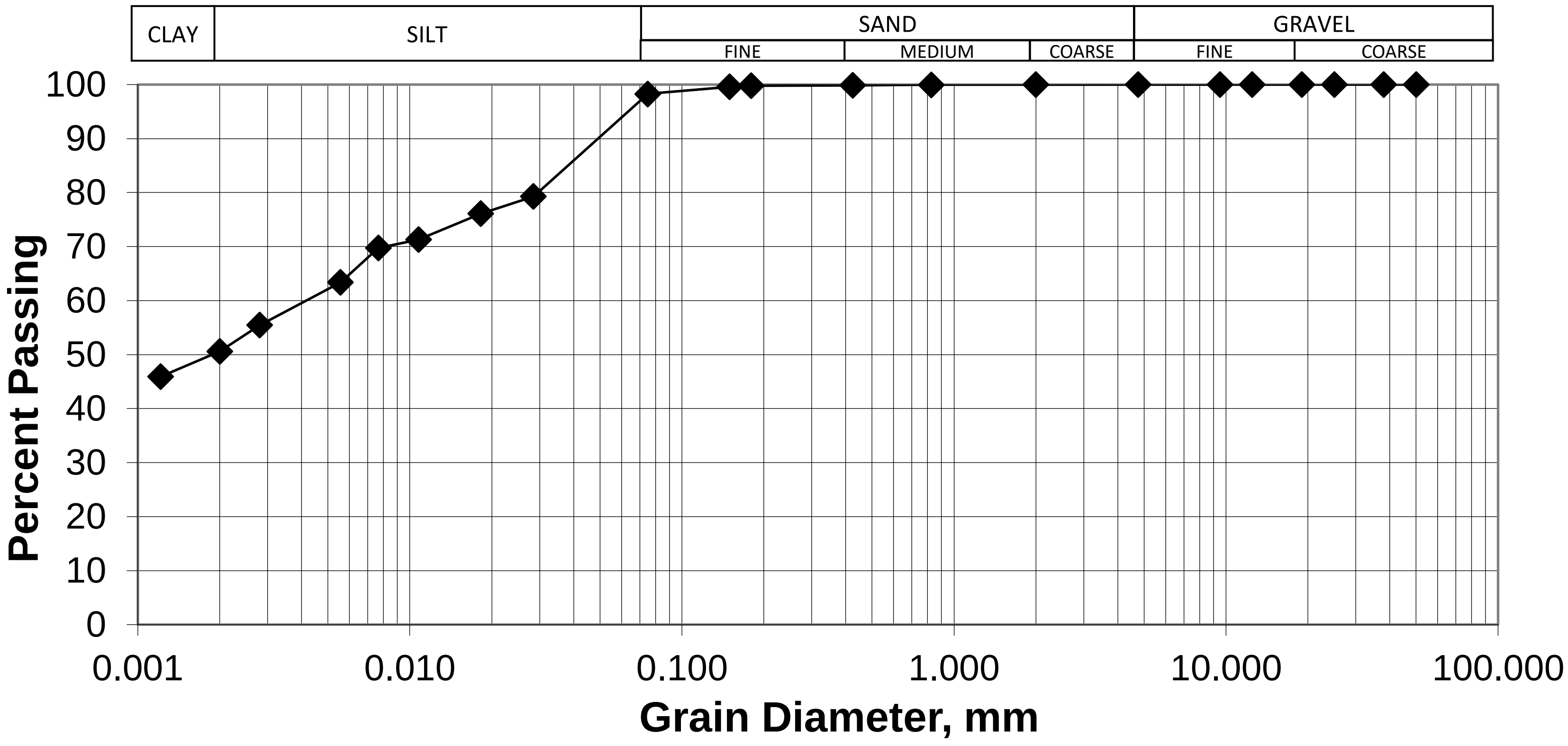
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	6-Jun-24
Sample Depth :	5.94 - 6.10 m	Lab Technician:	JEnriquez
Sample Number:	G7	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	98.3
38.0	100.0	2.00	100.0	0.0285	79.3
25.0	100.0	0.825	100.0	0.0183	76.1
19.0	100.0	0.425	99.9	0.0108	71.3
12.5	100.0	0.18	99.8	0.0077	69.7
9.5	100.0	0.15	99.6	0.0056	63.4
4.75	100.0	0.075	98.3	0.0028	55.5
				0.0020	50.6
				0.0012	45.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**47.6%**
Sand	**1.7%**	**Clay**	**50.6%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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Phone: 204 477 5381

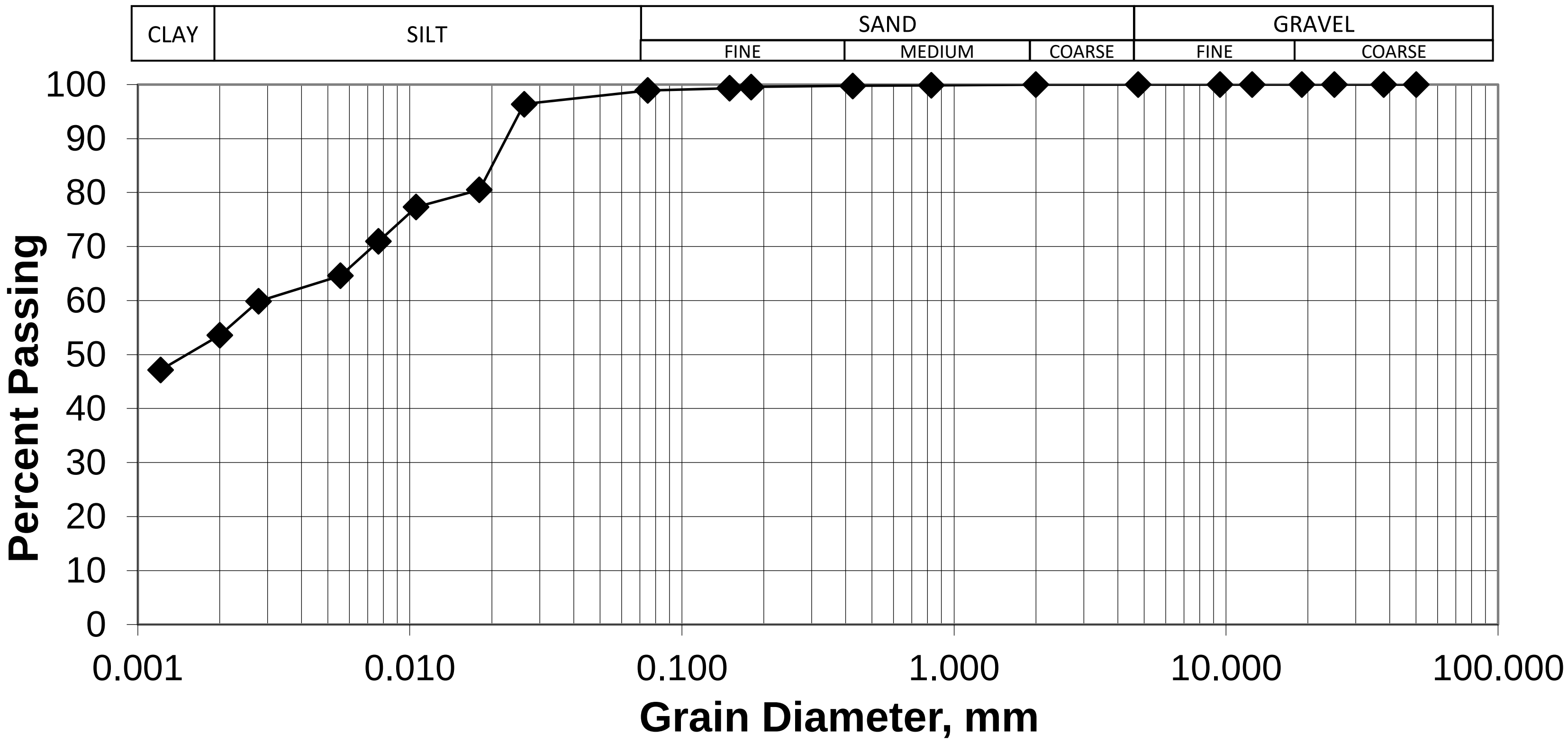
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	6-Jun-24
Sample Depth :	8.99 - 9.14 m	Lab Technician:	JEnriquez
Sample Number:	G10	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	98.9
38.0	100.0	2.00	100.0	0.0263	96.4
25.0	100.0	0.825	99.9	0.0180	80.5
19.0	100.0	0.425	99.8	0.0105	77.3
12.5	100.0	0.18	99.6	0.0077	71.0
9.5	100.0	0.15	99.3	0.0056	64.6
4.75	100.0	0.075	98.9	0.0028	59.9
				0.0020	53.5
				0.0012	47.1

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**45.3%**
Sand	**1.1%**	**Clay**	**53.5%**

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

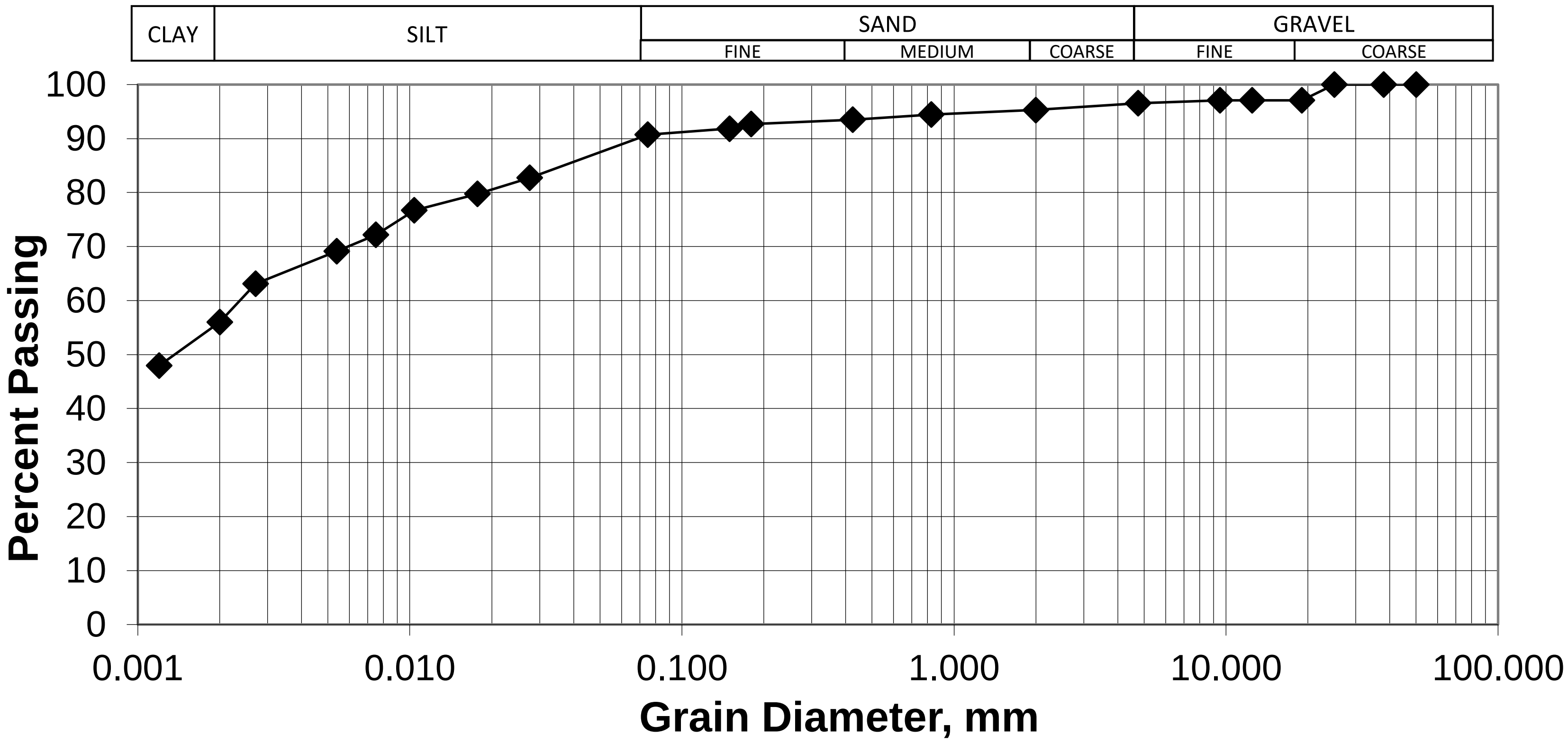
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	6-Jun-24
Sample Depth :	12.04 - 12.19 m	Lab Technician:	JEnriquez
Sample Number:	G13	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	96.6	0.0750	90.7
38.0	100.0	2.00	95.3	0.0276	86.9
25.0	100.0	0.825	94.5	0.0177	83.7
19.0	97.1	0.425	93.5	0.0104	80.5
12.5	97.1	0.18	92.7	0.0075	75.7
9.5	97.1	0.15	91.8	0.0054	72.6
4.75	96.6	0.075	90.7	0.0027	66.2
				0.0020	58.7
				0.0012	50.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**3.4%**	**Silt**	**32.0%**
Sand	**5.9%**	**Clay**	**58.7%**

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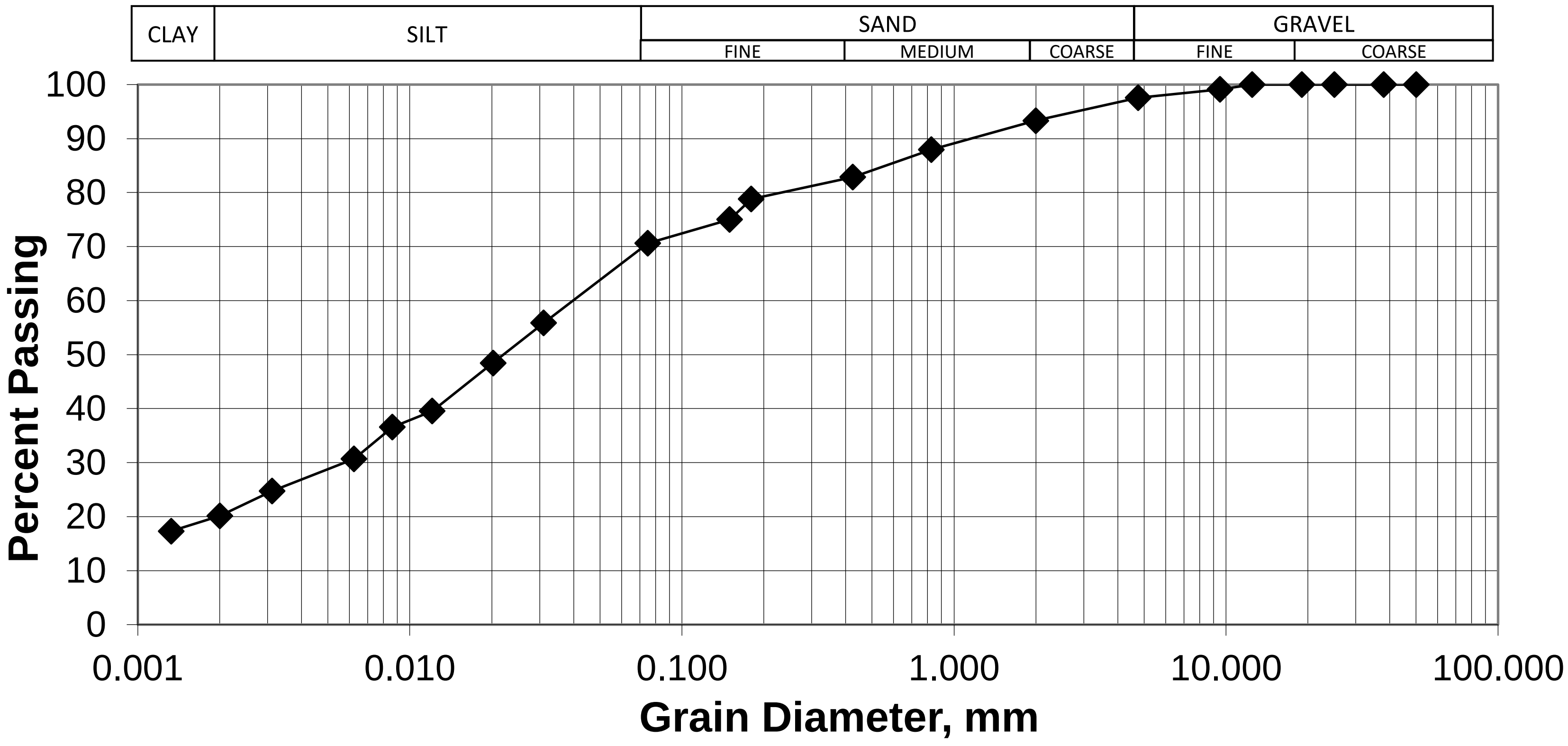
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-04	Sample Date:	6-Jun-24
Sample Depth :	12.95 - 13.11 m	Lab Technician:	JEnriquez
Sample Number:	G14	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	97.6	0.0750	70.6
38.0	100.0	2.00	93.3	0.0311	59.9
25.0	100.0	0.825	88.0	0.0202	51.9
19.0	100.0	0.425	82.9	0.0121	42.4
12.5	100.0	0.18	78.8	0.0086	39.2
9.5	99.1	0.15	75.0	0.0062	32.9
4.75	97.6	0.075	70.6	0.0031	26.5
				0.0020	21.5
				0.0013	18.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**2.4%**	**Silt**	**49.1%**
Sand	**26.9%**	**Clay**	**21.5%**

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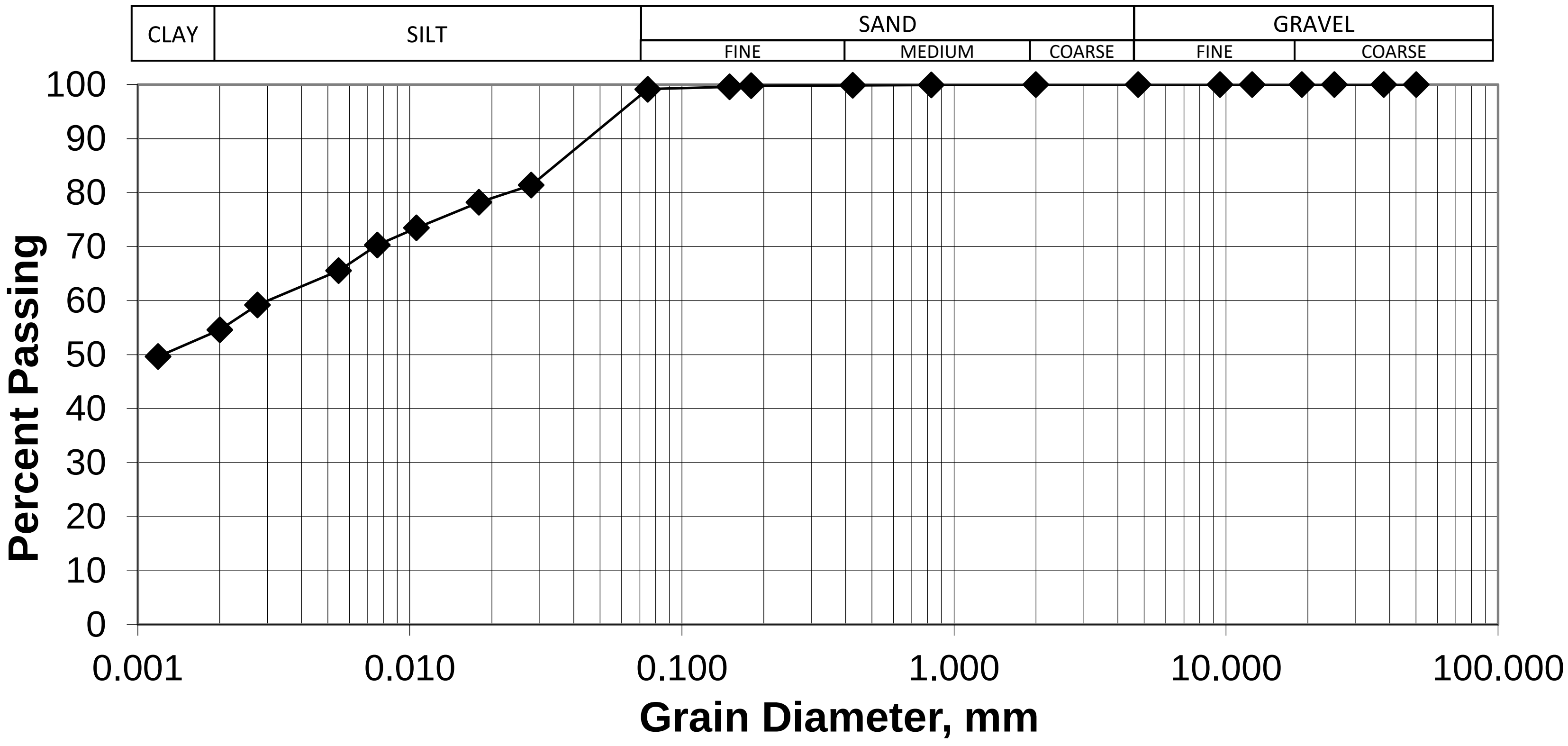
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	6-Jun-24
Sample Depth :	0.76 - 0.91 m	Lab Technician:	JEnriquez
Sample Number:	G2	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	99.1
38.0	100.0	2.00	100.0	0.0280	81.4
25.0	100.0	0.825	99.9	0.0179	78.2
19.0	100.0	0.425	99.9	0.0106	73.5
12.5	100.0	0.18	99.8	0.0076	70.3
9.5	100.0	0.15	99.6	0.0055	65.5
4.75	100.0	0.075	99.1	0.0028	59.2
				0.0020	54.6
				0.0012	49.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**44.6%**
Sand	**0.9%**	**Clay**	**54.6%**

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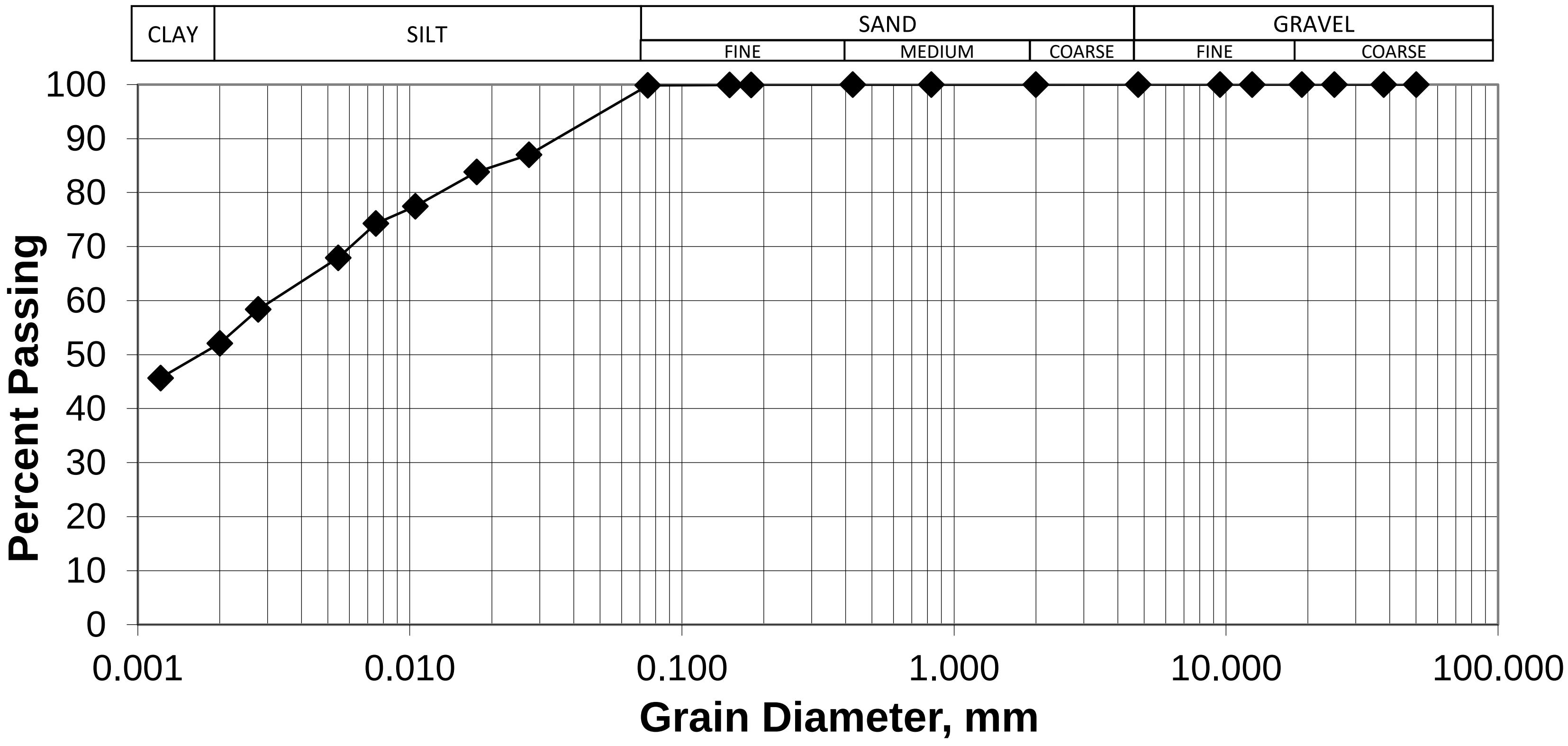
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	6-Jun-24
Sample Depth :	4.42 - 4.57 m	Lab Technician:	JEnriquez
Sample Number:	G6	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	99.9
38.0	100.0	2.00	100.0	0.0274	87.0
25.0	100.0	0.825	100.0	0.0176	83.8
19.0	100.0	0.425	100.0	0.0105	77.4
12.5	100.0	0.18	100.0	0.0075	74.3
9.5	100.0	0.15	99.9	0.0055	67.9
4.75	100.0	0.075	99.9	0.0028	58.4
				0.0020	52.1
				0.0012	45.7

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**47.8%**
Sand	**0.1%**	**Clay**	**52.1%**

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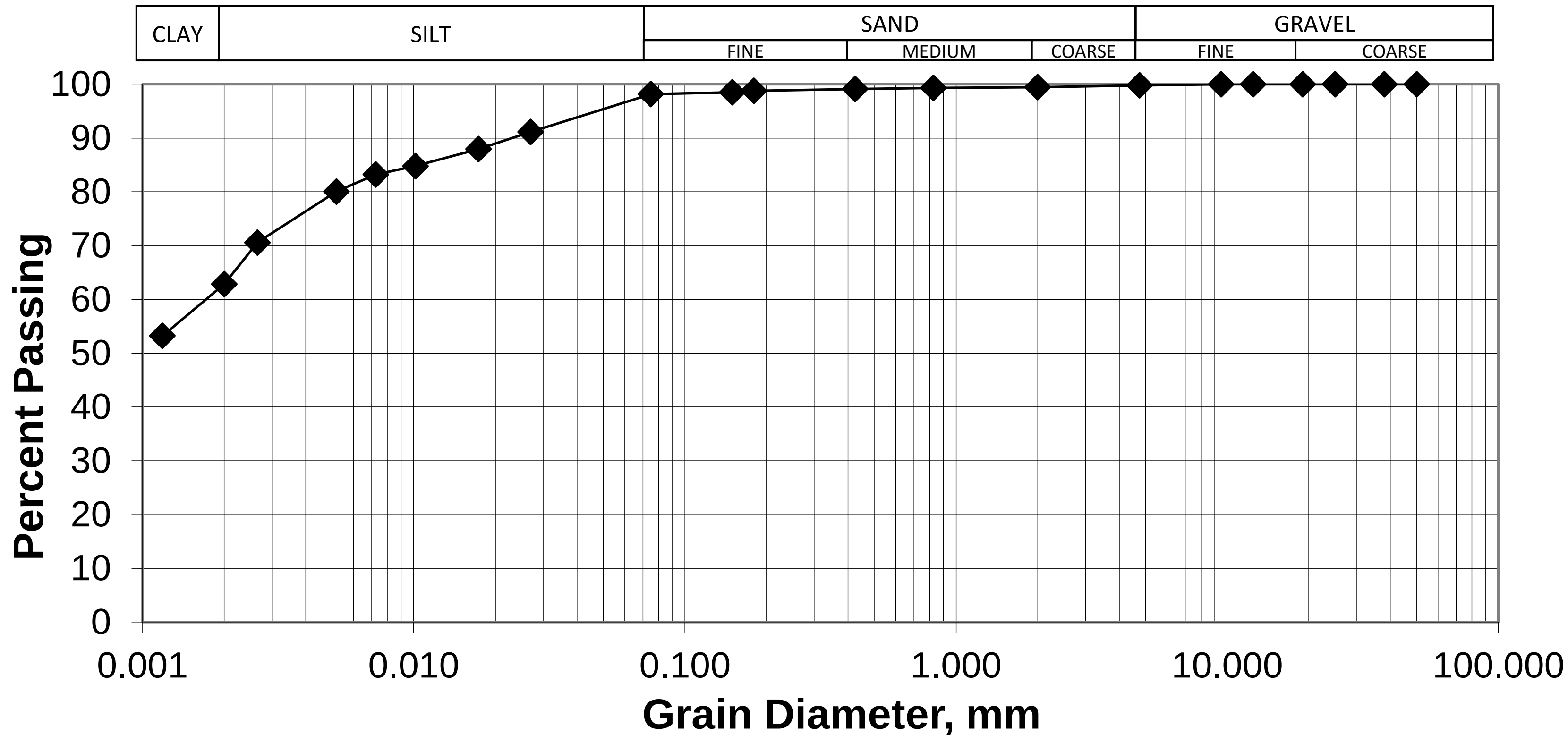
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	6-Jun-24
Sample Depth :	10.52 - 10.67 m	Lab Technician:	JEnriquez
Sample Number:	G12	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	99.8	0.0750	98.2
38.0	100.0	2.00	99.5	0.0270	91.6
25.0	100.0	0.825	99.3	0.0173	88.4
19.0	100.0	0.425	99.1	0.0102	85.3
12.5	100.0	0.18	98.8	0.0072	83.7
9.5	100.0	0.15	98.5	0.0052	80.5
4.75	99.8	0.075	98.2	0.0027	71.0
				0.0020	63.2
				0.0012	53.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.2%**	**Silt**	**35.0%**
Sand	**1.6%**	**Clay**	**63.2%**

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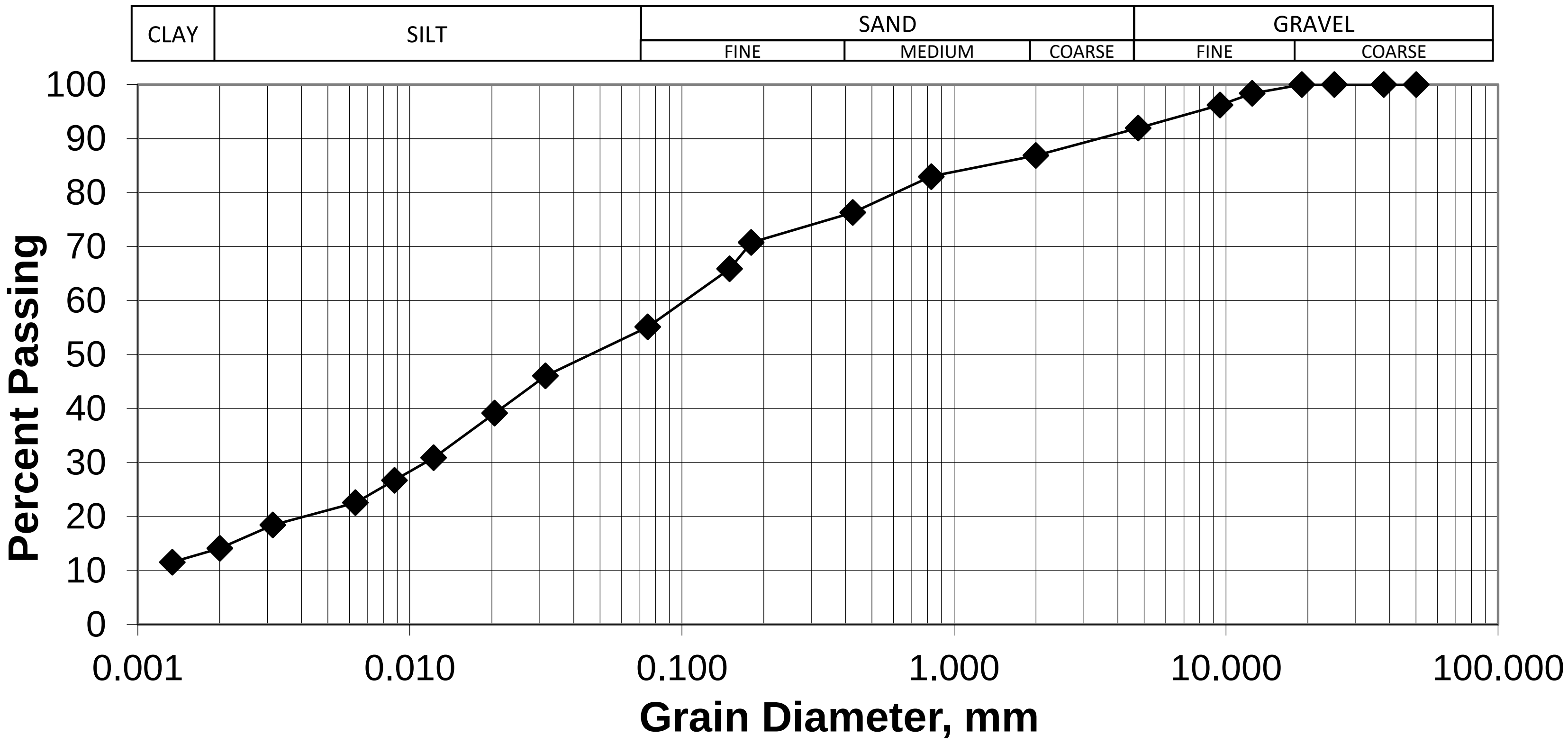
Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Supplier/Location:	Winnipeg, Manitoba
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	TH24-05	Sample Date:	6-Jun-24
Sample Depth :	13.56 - 13.72 m	Lab Technician:	JEnriquez
Sample Number:	G15	Date Tested:	11-Jun-24

Hydrometer (AASHTO T88)

Standard Test Methods for Particle Size Analysis of Soils

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	92.0	0.0750	55.1
38.0	100.0	2.00	86.9	0.0315	53.0
25.0	100.0	0.825	83.0	0.0205	45.1
19.0	100.0	0.425	76.3	0.0122	35.5
12.5	98.4	0.18	70.7	0.0088	30.8
9.5	96.2	0.15	65.9	0.0063	26.0
4.75	92.0	0.075	55.1	0.0031	21.2
				0.0020	16.2
				0.0013	13.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**8.0%**	**Silt**	**38.9%**
Sand	**36.8%**	**Clay**	**16.2%**

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Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 3, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 3, 2024
Sample Depth (m):	3.05 - 3.66 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	June 7, 2024
Sample Number:	T5	Tested By:	JEnriquez

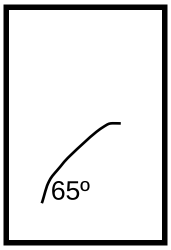
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, stiff, moist, silty, high plasticity, homogeneous

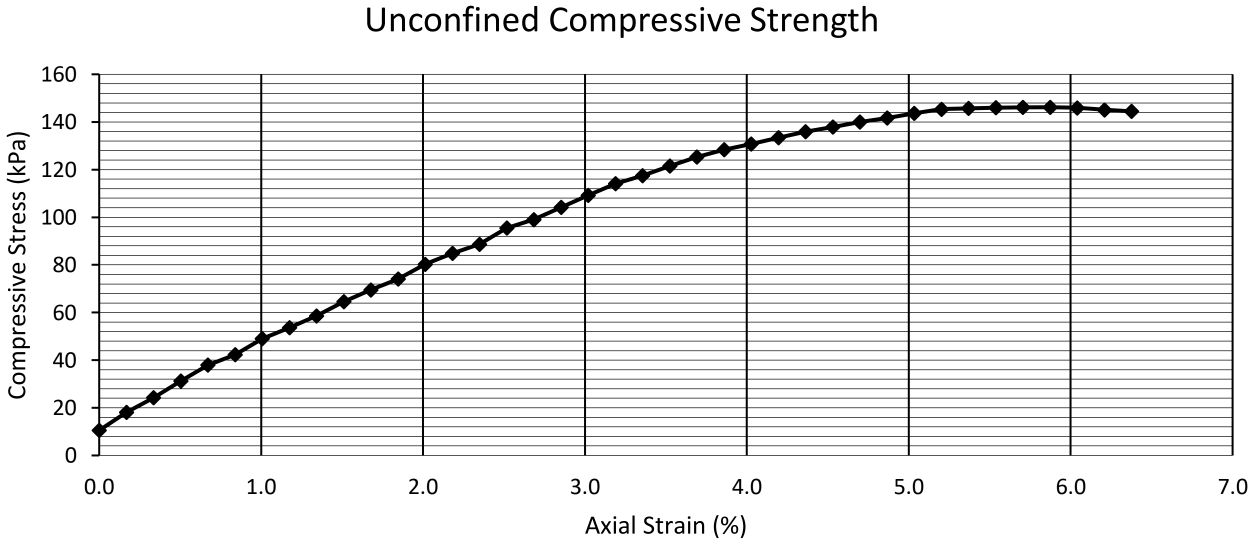
Average Diameter (cm):	7.17
Average Length (cm):	14.90
Length/Diameter Ratio:	2.08
Moisture content (%):	13.6
Bulk Density (g/cm³):	1.940
Bulk Unit Weight (kN/m³):	19.0
Bulk Unit Weight (pcf):	121.1
Dry Unit Weight (kN/m³):	16.74

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	34.3
Pocket Pen.	Undrained Shear Strength (kPa)	95.8

UCS	Unconfined compressive strength (kPa)	146.18	Undrained Shear Strength (kPa)	73.09
	Unconfined compressive strength (ksf)	3.053	Undrained Shear Strength (ksf)	1.526
	Avg. Rate of Strain to Failure (%/min):	1.01	Strain at Failure (%):	5.87



Comments:

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Laboratory Manager

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Geotechnical Discipline Lead

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Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 3, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 3, 2024
Sample Depth (m):	6.10 - 6.71 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	June 7, 2024
Sample Number:	T8	Tested By:	JEnriquez

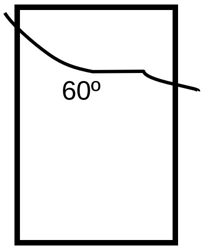
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, high plasticity, slickensided

Average Diameter (cm):	7.10
Average Length (cm):	14.73
Length/Diameter Ratio:	2.08
Moisture content (%):	15.0
Bulk Density (g/cm³):	1.797
Bulk Unit Weight (kN/m³):	17.6
Bulk Unit Weight (pcf):	112.2
Dry Unit Weight (kN/m³):	15.32

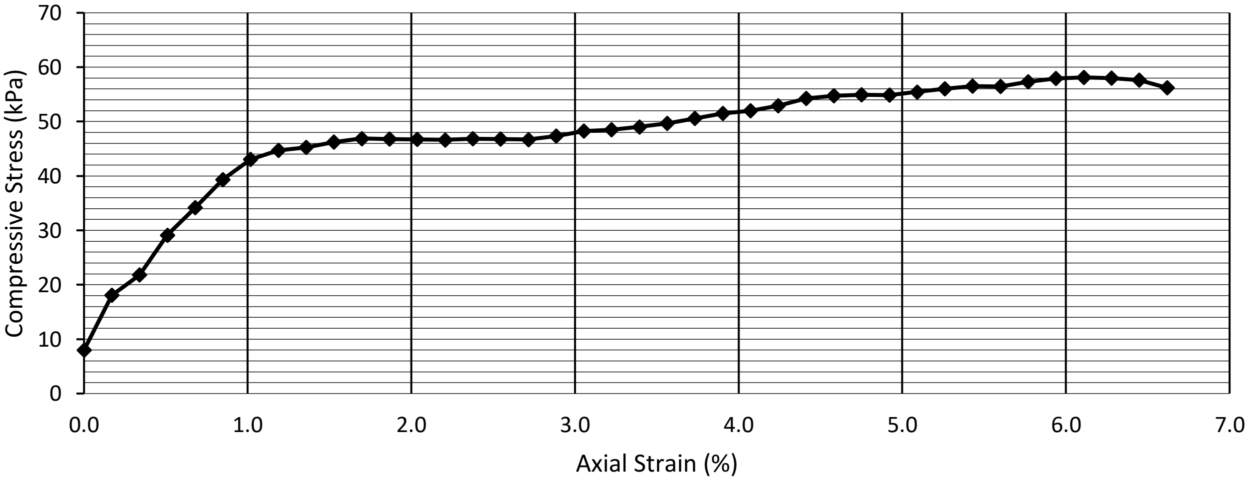
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	88.3
Pocket Pen.	Undrained Shear Strength (kPa)	48.7

UCS	Unconfined compressive strength (kPa)	58.12	Undrained Shear Strength (kPa)	29.06
	Unconfined compressive strength (ksf)	1.214	Undrained Shear Strength (ksf)	0.607
	Avg. Rate of Strain to Failure (%/min):	1.02	Strain at Failure (%):	6.11

Unconfined Compressive Strength



Comments:
Lower undrained shear strength (kPa) for unconfined compressive test due to the structure being slickensided.

Reviewed by:	Lee Boughton Laboratory Manager	Approved by:	German Leal, M.Eng., P.Eng. Geotechnical Discipline Lead

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Project Name: FGSV Siphon Replacement

Project Number:	60728226	Date Sampled:	June 3, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 3, 2024
Sample Depth (m):	12.19 - 12.80 m	Submitted By:	GAcurin
Sample Location:	TH24-01	Date Tested:	June 18, 2024
Sample Number:	T14	Tested By:	JEnriquez

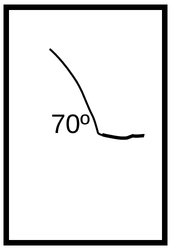
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.20
Average Length (cm):	14.40
Length/Diameter Ratio:	2.00
Moisture content (%):	47.3
Bulk Density (g/cm³):	1.725
Bulk Unit Weight (kN/m³):	16.9
Bulk Unit Weight (pcf):	107.7
Dry Unit Weight (kN/m³):	11.49

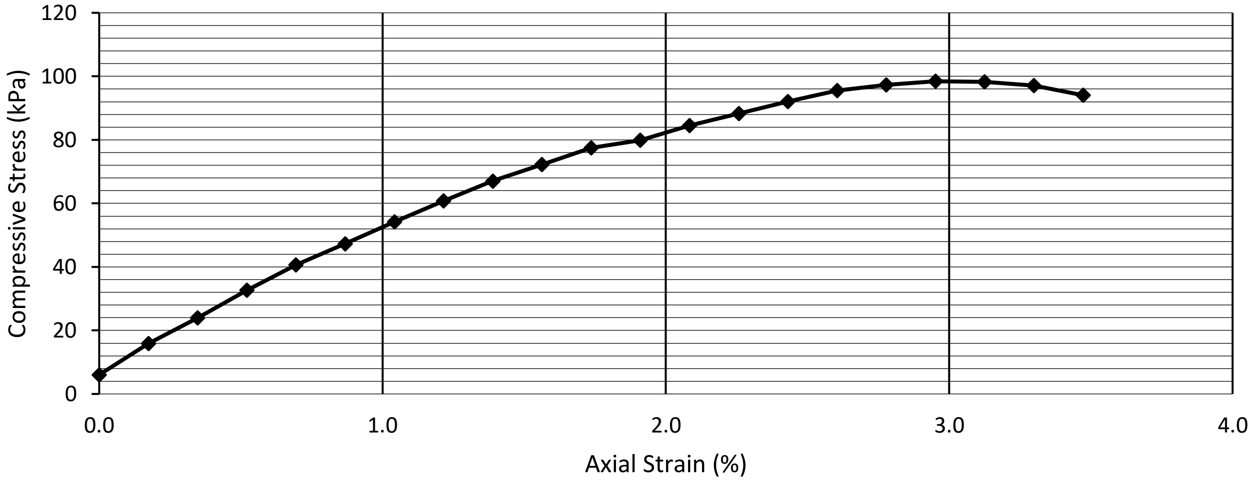
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	58.8
Pocket Pen.	Undrained Shear Strength (kPa)	47.9

UCS	Unconfined compressive strength (kPa)	98.45	Undrained Shear Strength (kPa)	49.23
	Unconfined compressive strength (ksf)	2.056	Undrained Shear Strength (ksf)	1.028
	Avg. Rate of Strain to Failure (%/min):	1.04	Strain at Failure (%):	2.95

Unconfined Compressive Strength



Comments:

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Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 4, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 4, 2024
Sample Depth (m):	3.05 - 3.66 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	June 18, 2024
Sample Number:	T5	Tested By:	JEnriquez

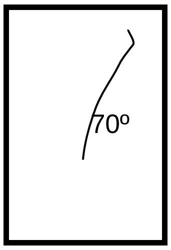
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.20
Average Length (cm):	13.90
Length/Diameter Ratio:	1.93
Moisture content (%):	33.4
Bulk Density (g/cm³):	1.884
Bulk Unit Weight (kN/m³):	18.5
Bulk Unit Weight (pcf):	117.6
Dry Unit Weight (kN/m³):	13.84

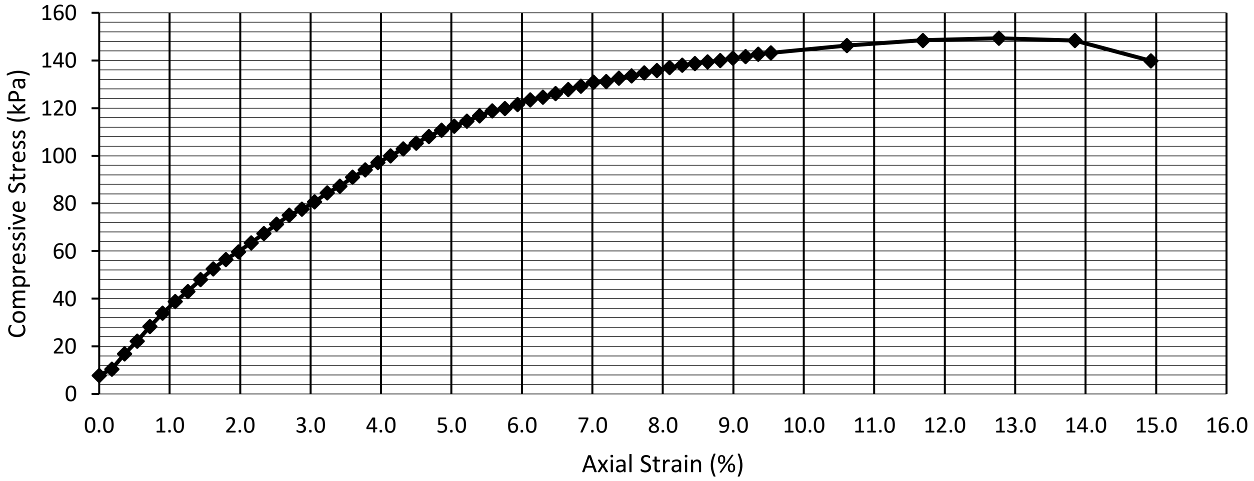
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	51.0
Pocket Pen.	Undrained Shear Strength (kPa)	30.3

UCS	Unconfined compressive strength (kPa)	149.31	Undrained Shear Strength (kPa)	74.65
	Unconfined compressive strength (ksf)	3.118	Undrained Shear Strength (ksf)	1.559
	Avg. Rate of Strain to Failure (%/min):	1.08	Strain at Failure (%):	12.77

Unconfined Compressive Strength



Comments:

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Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 4, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 4, 2024
Sample Depth (m):	9.14 - 9.75 m	Submitted By:	GAcurin
Sample Location:	TH24-02	Date Tested:	June 18, 2024
Sample Number:	T11	Tested By:	JEnriquez

Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

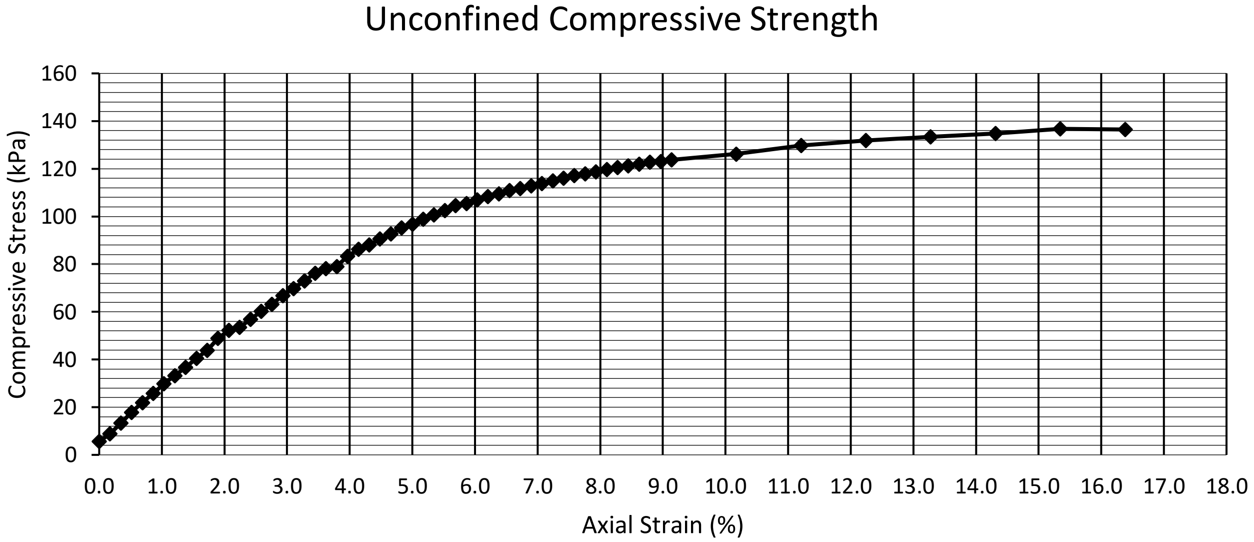
Soil Description:	CLAY - grey, stiff, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.07
Average Length (cm):	14.50
Length/Diameter Ratio:	2.05
Moisture content (%):	32.7
Bulk Density (g/cm³):	2.107
Bulk Unit Weight (kN/m³):	20.7
Bulk Unit Weight (pcf):	131.5
Dry Unit Weight (kN/m³):	15.57

FAILURE SKETCH

Torvane	Undrained Shear Strength (kPa)	49.0
Pocket Pen.	Undrained Shear Strength (kPa)	54.3

UCS	Unconfined compressive strength (kPa)	136.74	Undrained Shear Strength (kPa)	68.37
	Unconfined compressive strength (ksf)	2.856	Undrained Shear Strength (ksf)	1.428
	Avg. Rate of Strain to Failure (%/min):	1.03	Strain at Failure (%):	15.34



Comments:

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Laboratory Manager

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Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 6, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 6, 2024
Sample Depth (m):	3.05 - 3.66 m	Submitted By:	GAcurin
Sample Location:	TH24-04	Date Tested:	June 7, 2024
Sample Number:	T5	Tested By:	JEnriquez

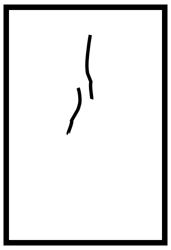
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, high plasticity, homogeneous

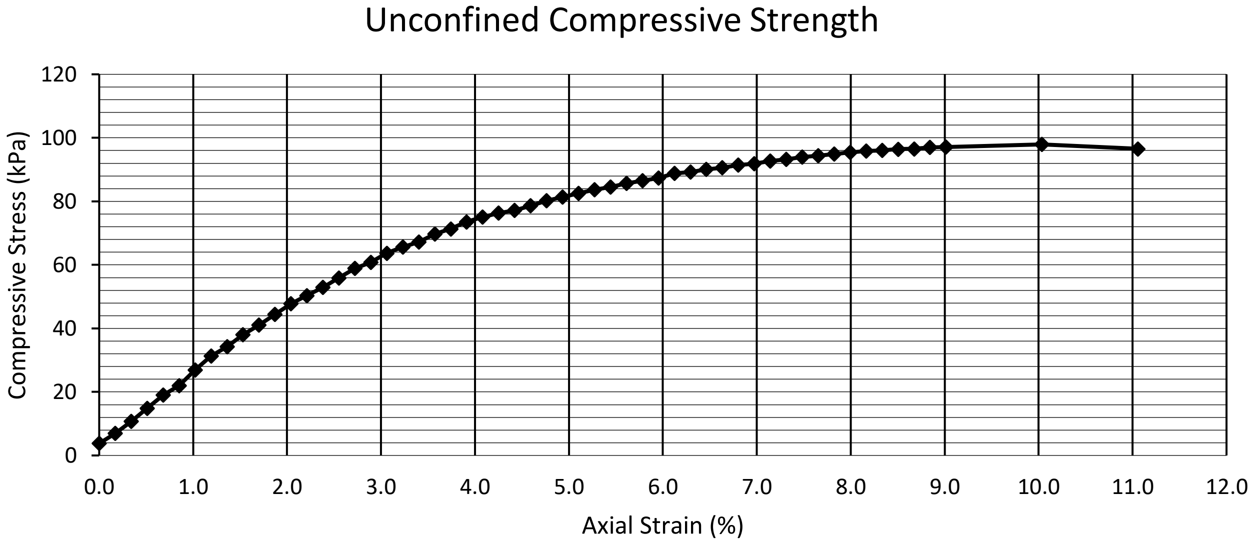
Average Diameter (cm):	7.10
Average Length (cm):	14.70
Length/Diameter Ratio:	2.07
Moisture content (%):	14.6
Bulk Density (g/cm³):	1.936
Bulk Unit Weight (kN/m³):	19.0
Bulk Unit Weight (pcf):	120.9
Dry Unit Weight (kN/m³):	16.57

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	66.7
Pocket Pen.	Undrained Shear Strength (kPa)	39.9

UCS	Unconfined compressive strength (kPa)	97.93	Undrained Shear Strength (kPa)	48.97
	Unconfined compressive strength (ksf)	2.045	Undrained Shear Strength (ksf)	1.023
	Avg. Rate of Strain to Failure (%/min):	1.02	Strain at Failure (%):	10.03



Comments:

Reviewed by:	Lee Boughton	Approved by:	German Leal, M.Eng., P.Eng.
	Laboratory Manager		Geotechnical Discipline Lead

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 6, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 6, 2024
Sample Depth (m):	9.14 - 9.75 m	Submitted By:	GAcurin
Sample Location:	TH24-04	Date Tested:	June 18, 2024
Sample Number:	T11	Tested By:	JEnriquez

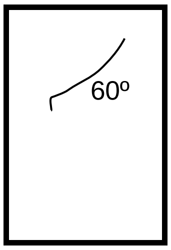
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.10
Average Length (cm):	15.60
Length/Diameter Ratio:	2.20
Moisture content (%):	33.1
Bulk Density (g/cm³):	1.961
Bulk Unit Weight (kN/m³):	19.2
Bulk Unit Weight (pcf):	122.4
Dry Unit Weight (kN/m³):	14.45

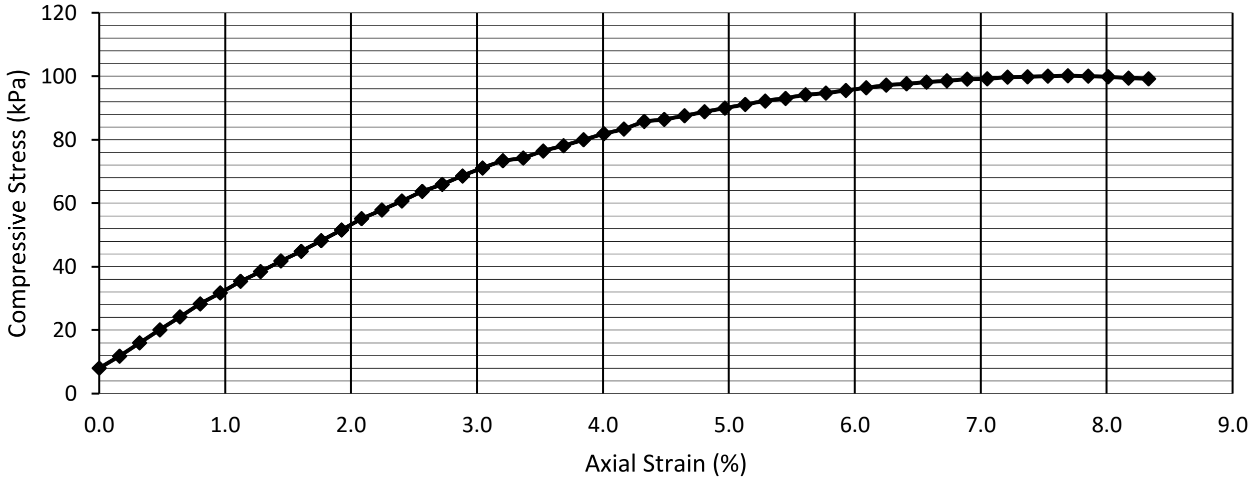
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	39.2
Pocket Pen.	Undrained Shear Strength (kPa)	39.9

UCS	Unconfined compressive strength (kPa)	100.19	Undrained Shear Strength (kPa)	50.09
	Unconfined compressive strength (ksf)	2.092	Undrained Shear Strength (ksf)	1.046
	Avg. Rate of Strain to Failure (%/min):	0.96	Strain at Failure (%):	7.69

Unconfined Compressive Strength



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 5, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 5, 2024
Sample Depth (m):	1.52 - 2.13 m	Submitted By:	GAcurin
Sample Location:	TH24-05	Date Tested:	June 7, 2024
Sample Number:	T4	Tested By:	JEnriquez

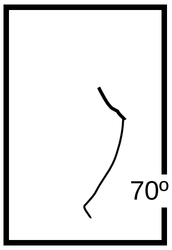
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - brown, stiff, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.20
Average Length (cm):	15.00
Length/Diameter Ratio:	2.08
Moisture content (%):	14.2
Bulk Density (g/cm³):	1.912
Bulk Unit Weight (kN/m³):	18.8
Bulk Unit Weight (pcf):	119.4
Dry Unit Weight (kN/m³):	16.42

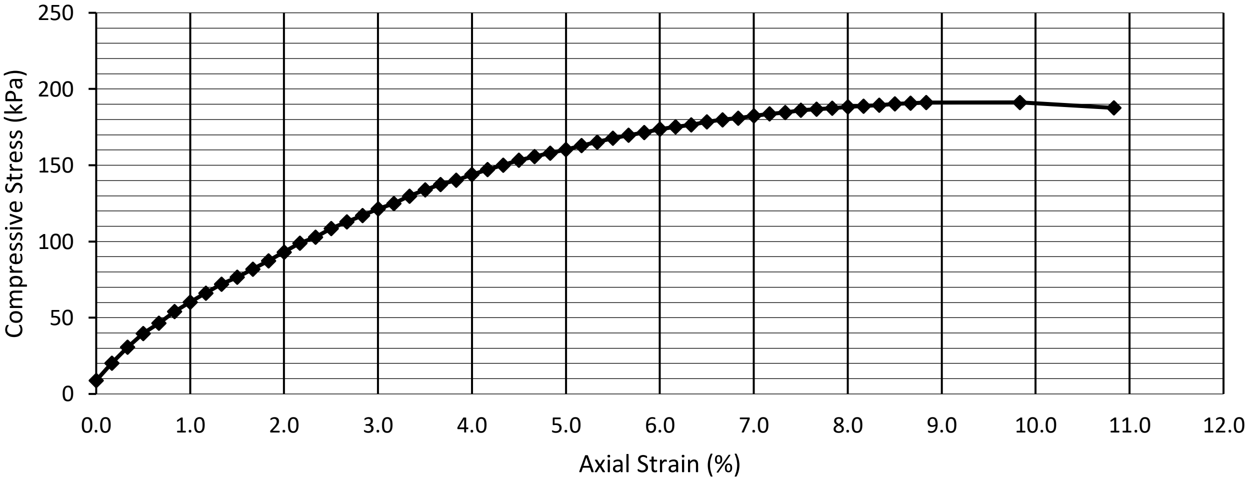
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	83.4
Pocket Pen.	Undrained Shear Strength (kPa)	79.8

UCS	Unconfined compressive strength (kPa)	191.25	Undrained Shear Strength (kPa)	95.63
	Unconfined compressive strength (ksf)	3.994	Undrained Shear Strength (ksf)	1.997
	Avg. Rate of Strain to Failure (%/min):	1.00	Strain at Failure (%):	9.83

Unconfined Compressive Strength



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 5, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 5, 2024
Sample Depth (m):	7.62 - 8.23 m	Submitted By:	GAcurin
Sample Location:	TH24-05	Date Tested:	June 18, 2024
Sample Number:	T10	Tested By:	JEnriquez

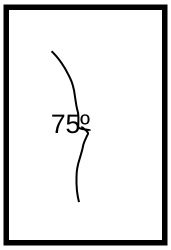
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, stiff, moist, silty, high plasticity, homogeneous

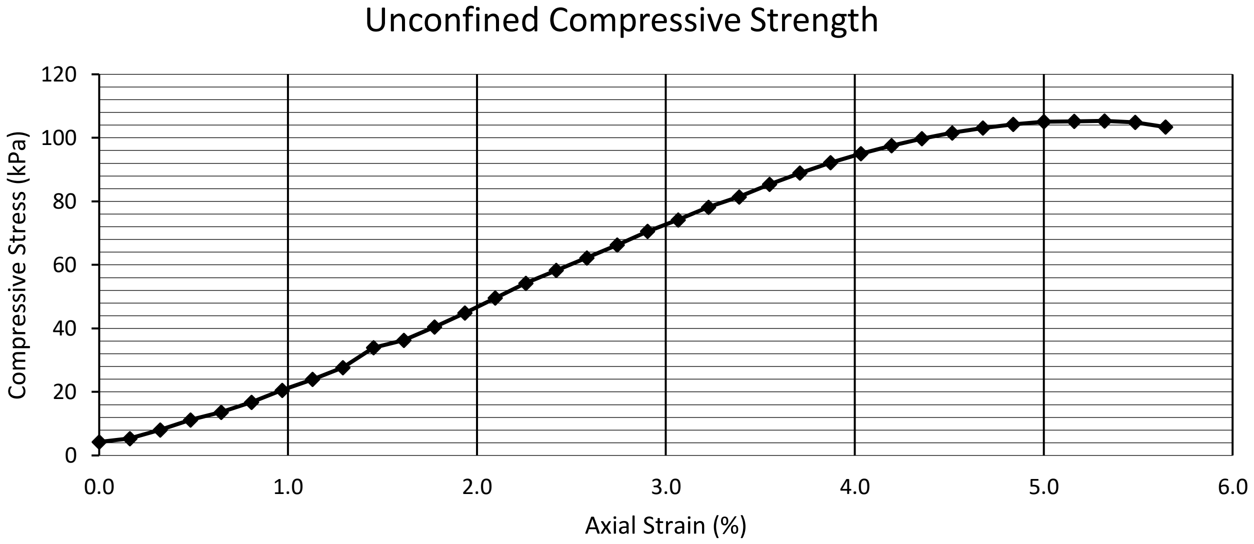
Average Diameter (cm):	7.07
Average Length (cm):	15.50
Length/Diameter Ratio:	2.19
Moisture content (%):	32.1
Bulk Density (g/cm³):	2.020
Bulk Unit Weight (kN/m³):	19.8
Bulk Unit Weight (pcf):	126.1
Dry Unit Weight (kN/m³):	14.99

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	66.7
Pocket Pen.	Undrained Shear Strength (kPa)	54.3

UCS	Unconfined compressive strength (kPa)	105.34	Undrained Shear Strength (kPa)	52.67
	Unconfined compressive strength (ksf)	2.200	Undrained Shear Strength (ksf)	1.100
	Avg. Rate of Strain to Failure (%/min):	0.97	Strain at Failure (%):	5.32



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

AECOM

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Winnipeg Geotechnical Laboratory
99 Commerce Drive, Winnipeg, MB R3P 0Y7
Phone: 204 477 5381

Project Name:	FGSV Siphon Replacement		
Project Number:	60728226	Date Sampled:	June 5, 2024
Client:	City Of WInnipeg	Sampled By:	GAcurin
Supplier/Location:	Winnipeg, MB	Date Received:	June 5, 2024
Sample Depth (m):	10.67 - 11.28 m	Submitted By:	GAcurin
Sample Location:	TH24-05	Date Tested:	June 7, 2024
Sample Number:	T13	Tested By:	JEnriquez

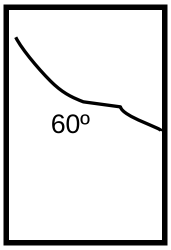
Unconfined Compressive Strength (ASTM D2166)

Standard Test Method for Unconfined Compressive Strenght of Cohesive Soil, using strain-controlled application of the axial load.

Soil Description:	CLAY - grey, firm, moist, silty, high plasticity, homogeneous

Average Diameter (cm):	7.10
Average Length (cm):	14.80
Length/Diameter Ratio:	2.08
Moisture content (%):	16.1
Bulk Density (g/cm³):	1.811
Bulk Unit Weight (kN/m³):	17.8
Bulk Unit Weight (pcf):	113.1
Dry Unit Weight (kN/m³):	15.31

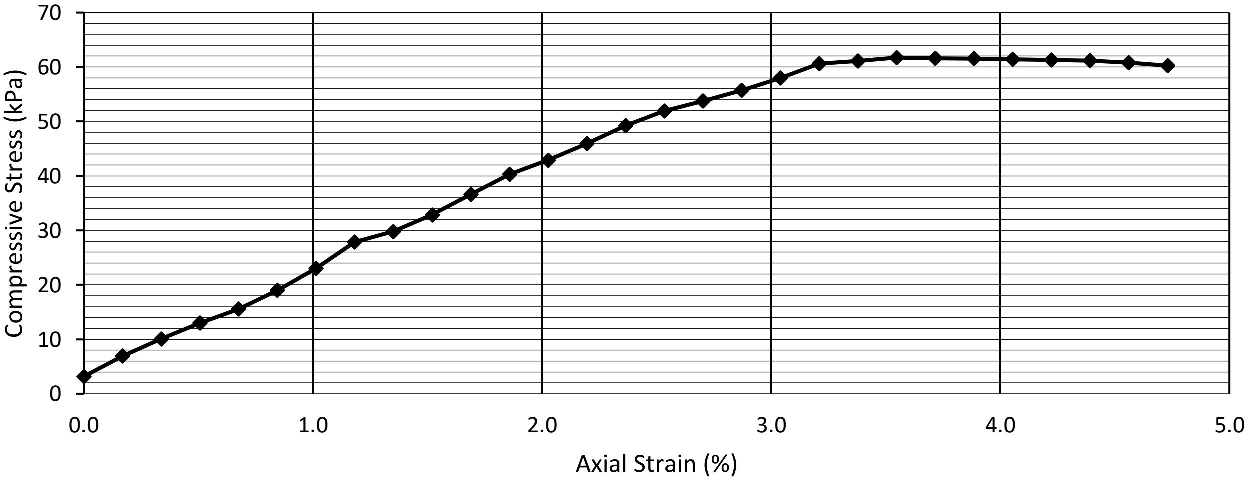
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	44.1
Pocket Pen.	Undrained Shear Strength (kPa)	23.9

UCS	Unconfined compressive strength (kPa)	61.74	Undrained Shear Strength (kPa)	30.87
	Unconfined compressive strength (ksf)	1.289	Undrained Shear Strength (ksf)	0.645
	Avg. Rate of Strain to Failure (%/min):	1.01	Strain at Failure (%):	3.55

Unconfined Compressive Strength



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead

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99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	FGSV Siphon Replacement	Supplier:	AECOM
Project Number:	60728226	Specification:	N/A
Client:	City Of Winnipeg	Field Technician:	GAcurin
Sample Location:	Winnipeg, Manitoba	Sample Date:	June 6, 2024
Sample Depth:	Varies	Lab Technician:	JEnriquez
Sample Number:	Varies	Date Tested:	June 6, 2024

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**
TH24-01	G1	0.15 - 0.30 m	**31.4%**
TH24-01	G2	0.61 - 0.76 m	**34.7%**
TH24-01	G3	1.37 - 1.52 m	**29.9%**
TH24-01	G4	2.90 - 3.05 m	**27.9%**
TH24-01	G6	4.42 - 4.57 m	**42.0%**
TH24-01	G7	5.94 - 6.10 m	**43.7%**
TH24-01	G9	7.47 - 7.62 m	**49.6%**
TH24-01	G10	8.99 - 9.14 m	**44.5%**
TH24-01	G12	10.52 - 10.67 m	**49.3%**
TH24-01	G13	12.04 - 12.19 m	**46.5%**
TH24-01	G15	13.56 - 13.72 m	**51.3%**
TH24-01	G16	15.09 - 15.24 m	**50.1%**
TH24-01	G17	16.61 - 16.76 m	**13.8%**
		0.00 - 0.00 m	**-**
TH24-02	G1	0.15 - 0.30 m	**33.3%**
TH24-02	G2	0.61 - 0.76 m	**33.9%**
TH24-02	G3	1.37 - 1.52 m	**35.0%**
TH24-02	G4	2.90 - 3.05 m	**34.9%**
TH24-02	G6	4.42 - 4.57 m	**33.6%**
TH24-02	G7	5.94 - 6.10 m	**33.8%**
TH24-02	G9	7.47 - 7.62 m	**36.5%**
TH24-02	G10	8.99 - 9.14 m	**38.7%**
TH24-02	G12	10.52 - 10.67 m	**48.0%**
TH24-02	G13	12.04 - 12.19 m	**12.7%**
TH24-02	G14	12.80 - 12.95 m	**13.1%**
		0.00 - 0.00 m	**-**
TH24-04	G1	0.15 - 0.30 m	**32.8%**
TH24-04	G2	0.61 - 0.76 m	**35.0%**
TH24-04	G3	1.37 - 1.52 m	**35.6%**
TH24-04	G4	2.90 - 3.05 m	**31.2%**
TH24-04	G6	4.42 - 4.57 m	**32.4%**
TH24-04	G7	5.94 - 6.10 m	**38.1%**
TH24-04	G9	7.47 - 7.62 m	**42.0%**
TH24-04	G10	8.99 - 9.14 m	**32.7%**
TH24-04	G12	10.52 - 10.67 m	**38.4%**
TH24-04	G13	12.04 - 12.19 m	**39.7%**
TH24-04	G14	12.95 - 13.11 m	**18.5%**
		0.00 - 0.00 m	**-**

Location	Sample	Depth (m)	**Moisture Content (%)**
TH24-05	G1	0.15 - 0.30 m	**35.6%**
TH24-05	G2	0.76 - 0.91 m	**35.6%**
TH24-05	G3	1.37 - 1.52 m	**33.2%**
TH24-05	G5	2.90 - 3.05 m	**31.2%**
TH24-05	G6	4.42 - 4.57 m	**31.2%**
TH24-05	G8	5.94 - 6.10 m	**32.3%**
TH24-05	G9	7.47 - 7.62 m	**31.5%**
TH24-05	G11	8.99 - 9.14 m	**39.3%**
TH24-05	G12	10.52 - 10.67 m	**44.4%**
TH24-05	G14	12.04 - 12.19 m	**39.7%**
TH24-05	G15	13.56 - 13.72 m	**15.5%**
TH24-05	G16	14.48 - 14.63 m	**11.4%**
		0.00 - 0.00 m	**-**
		0.00 - 0.00 m	**-**
		0.00 - 0.00 m	**-**
		0.00 - 0.00 m	**-**
		0.00 - 0.00 m	**-**
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www.eng-tech.ca

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

Ref. No.: 24-27-1-8,9**R1**

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Submitted By: Client
Date Cored: -
Received By: ENG-TECH (Kevin Dowbeta)
Core Conditioning: As received moisture condition
Specimen Temperature: 24.0°C (room temperature)

Page: 1 of 1
Date Received: Aug 1/24
Tested By: ENG-TECH (Kevin Dowbeta)
Method: ASTM D2938-95

Core No.	Client ID	Test Hole Location / Core Depth (m)	Length		Average Diameter (mm)	Rate of Loading (kN/s)	Compressive Strength (MPa)	Date Tested (m/d/y)
			Cored (mm)	Tested (mm)				
1	C18	TH24-01, 18.3 - 18.5	191	157.25	63.00	0.7	78	Aug 7/24
2	C23	TH24-05, 23.75 - 24.2	445	136.50	63.00	0.7	128	Aug 7/24

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Revision 1: Core No. 2 Client ID

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

Enclosure: Unconfined Compressive Strength Of Intact Rock Core Specimen Reports
Ref. No.'s 24-27-1-8 and 9

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

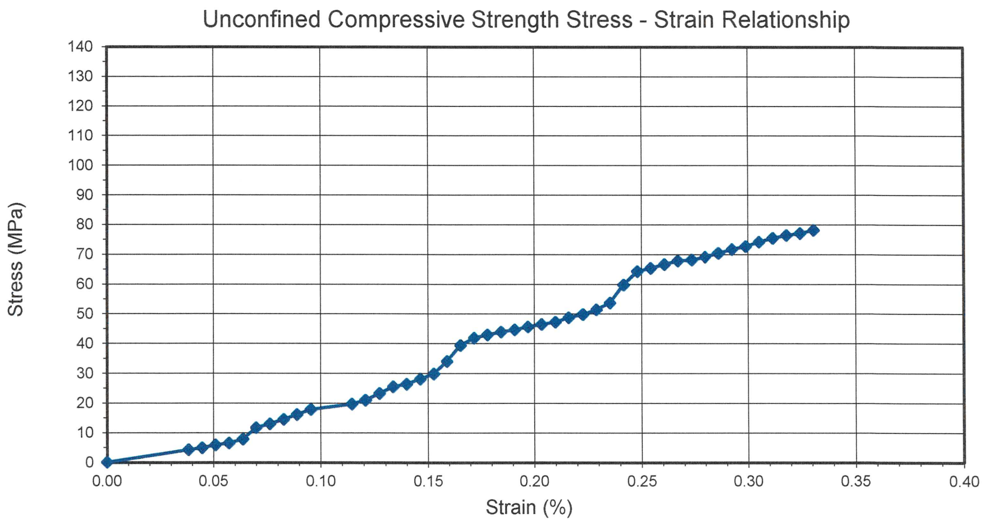
Ref. No.: 24-27-1-8

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D. C18
Test Hole/Depth TH24-01, 18.3 - 18.5 meters
Date Cored: -
Date Received: Aug 1/24
Compression Machine Model: Soil Test CT-710

Submitted By: Client
Date Tested: Aug 7/24
Tested By: ENG-TECH (Kevin Dowbeta)
Method: ASTM D2938-95


Unconfined Compressive Strength Stress - Strain Relationship

Test Data

Specimen Moisture Condition: As received moisture
Specimen Temperature: 24°C
Average Length of Specimen: 157.25 mm
Average Diameter of Specimen: 63.00 mm
Load Rate: 0.7 kN/s
Maximum Load: 243.3 kN
Compressive Strength: 78 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

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Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

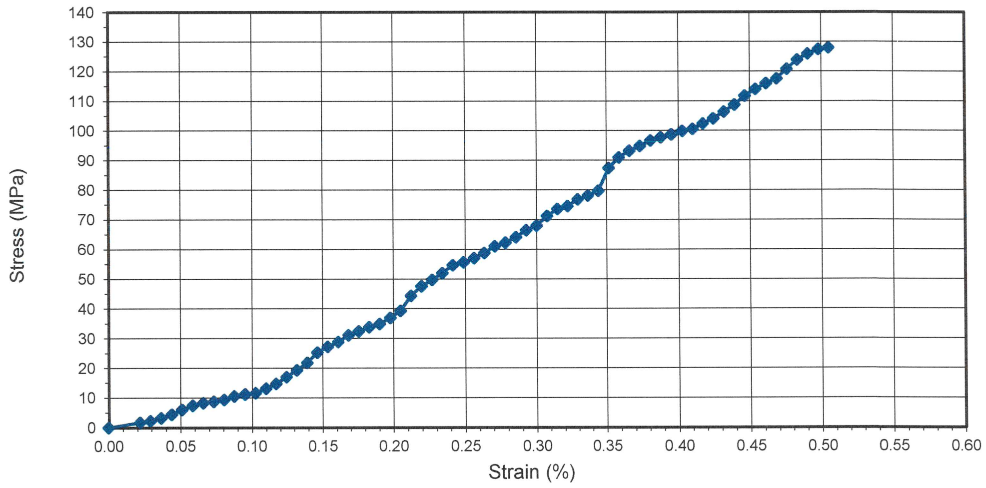
Ref. No.: 24-27-1-9**R1**

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D.	C23		TH24-05, 23.75 - 24.2
Test Hole/Depth	TH24-05, 23.75 – 24.2 meters	Submitted By:	Client
Date Cored:	-	Date Tested:	Aug 7/24
Date Received:	Aug 1/24	Tested By:	ENG-TECH (Kevin Dowbeta)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95

Unconfined Compressive Strength Stress - Strain Relationship



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	136.50 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 0.7 kN/s	Maximum Load: 398.5 kN	Compressive Strength:	128 MPa

Comments:

Revision 1: Test Hole, Depth

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
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R3P 1J9

File No.: 24-027-01

Ref. No.: 24-27-1-10,11,12

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Submitted By: Client
Date Cored: -
Received By: ENG-TECH (Jessica Bauer)
Core Conditioning: As received moisture condition
Specimen Temperature: 24.0°C (room temperature)

Page: 1 of 1
Date Received: Aug 16/24
Tested By: ENG-TECH (Kyle Zebiere)
Method: ASTM D2938-95

Core No.	Client ID	Test Hole Location / Core Depth (m)	Length		Average Diameter (mm)	Rate of Loading (kN/s)	Compressive Strength (MPa)	Date Tested (m/d/y)
			Cored (mm)	Tested (mm)				
1	C20	TH24-03, 29.97 - 30.19	210	140.00	63.00	0.7	87.7	Aug 22/24
2	C21	TH24-03, 31.43 - 31.65	212	154.00	63.00	0.7	50.6	Aug 22/24
3	C22	TH24-03, 32.28 - 32.76	470	155.50	63.00	0.7	35.3	Aug 22/24

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

Enclosure: Unconfined Compressive Strength of Intact Rock Core Specimen Reports
Ref. No.'s 24-27-1-10, 11 and 12

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

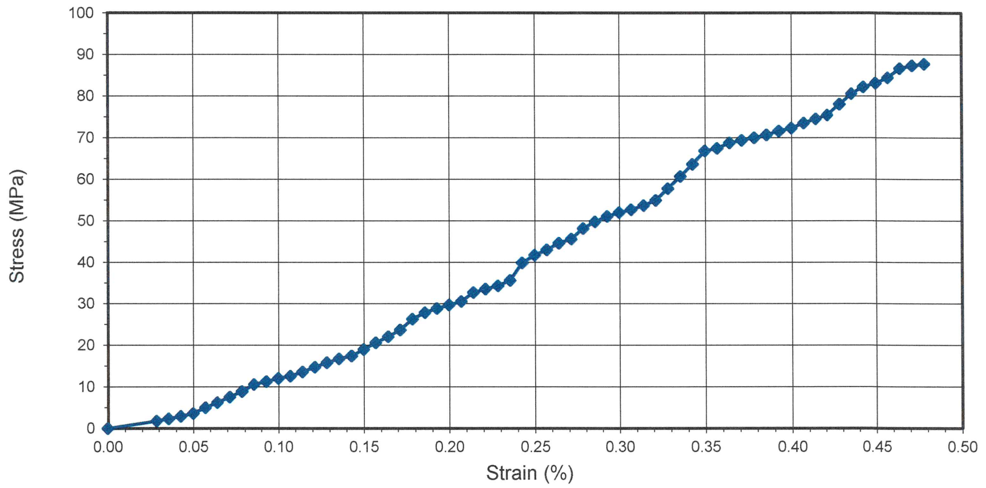
File No.: 24-027-01
Ref. No.: 24-27-1-10

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D. C20
Test Hole/Depth TH24-03, 29.97 - 30.19 meters
Date Cored: -
Date Received: Aug 16/24
Compression Machine Model: Soil Test CT-710
Submitted By: Client
Date Tested: Aug 22/24
Tested By: ENG-TECH (Kyle Zebiere)
Method: ASTM D2938-95

Unconfined Compressive Strength Stress - Strain Relationship



Test Data

Specimen Moisture Condition: As received moisture
Specimen Temperature: 24.0°C
Average Length of Specimen: 140.00 mm
Average Diameter of Specimen: 63.00 mm
Load Rate: 0.7 kN/s
Maximum Load: 273.4 kN
Compressive Strength: 87.7 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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R2J 3W8
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www.eng-tech.ca

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

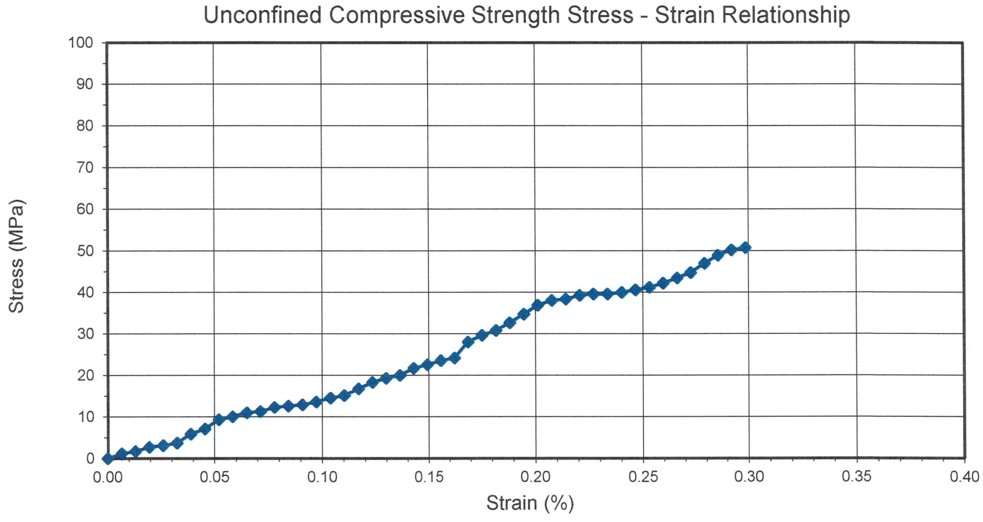
AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01
Ref. No.: 24-27-1-11

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D.	C21		
Test Hole/Depth	TH24-03, 31.43 - 31.65 meters	Submitted By:	Client
Date Cored:	-	Date Tested:	Aug 22/24
Date Received:	Aug 16/24	Tested By:	ENG-TECH (Kyle Zebiere)
Compression Machine Model:	Soil Test CT-710	Method:	ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	24°C
Average Length of Specimen:	154.00 mm	Average Diameter of Specimen:	63.00 mm
Load Rate: 0.7 kN/s	Maximum Load: 157.7 Kn	Compressive Strength:	50.6 MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

Ref. No.: 24-27-1-12

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D. C22

Test Hole/Depth TH24-03, 32.28 - 32.76 meters

Date Cored: -

Date Received: Aug 16/24

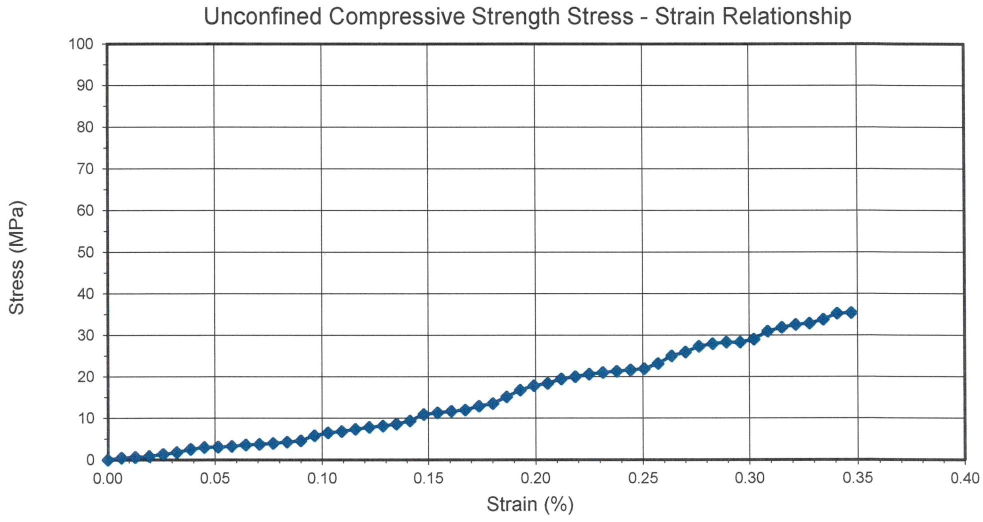
Compression Machine Model: Soil Test CT-710

Submitted By: Client

Date Tested: Aug 22/24

Tested By: ENG-TECH (Kyle Zebiere)

Method: ASTM D2938-95



Unconfined Compressive Strength Stress - Strain Relationship

Test Data

Specimen Moisture Condition:	As received moisture		Specimen Temperature:	24°C	
Average Length of Specimen:	155.50 mm		Average Diameter of Specimen:	63.00	mm
Load Rate: 0.7 kN/s	Maximum Load:	110.0 kN	Compressive Strength:	35.3	MPa

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

File No.: 24-027-01

Ref. No.: 24-27-1-19, 20

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Submitted By: Client
Date Cored: Aug 13/24
Received By: ENG-TECH (Rey Batac)
Core Conditioning: As received moisture condition
Specimen Temperature: 23.0°C (room temperature)

Page: 1 of 1
Date Received: Feb 7/25
Tested By: ENG-TECH (Kyle Zebiere)
Method: ASTM D2938-95

Core No.	Client ID	Test Hole Location / Core Depth (m)	Length		Average Diameter (mm)	Rate of Loading (kN/s)	Compressive Strength (MPa)	Date Tested (m/d/y)
			Cored (mm)	Tested (mm)				
1	C09	TH24-03, 53'5.5" - 54'1.5"	198	134.50	63.25	0.12	93	Feb 14/25
2	C10	TH24-03, 57'3.5" - 58'1.5"	248	156.50	63.00	0.12	235	Feb 14/25

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Deviation from test procedure: none

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

Enclosure: Unconfined Compressive Strength of Intact Rock Core Specimen Reports
Ref. No.'s 24-27-1-19 and 20

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

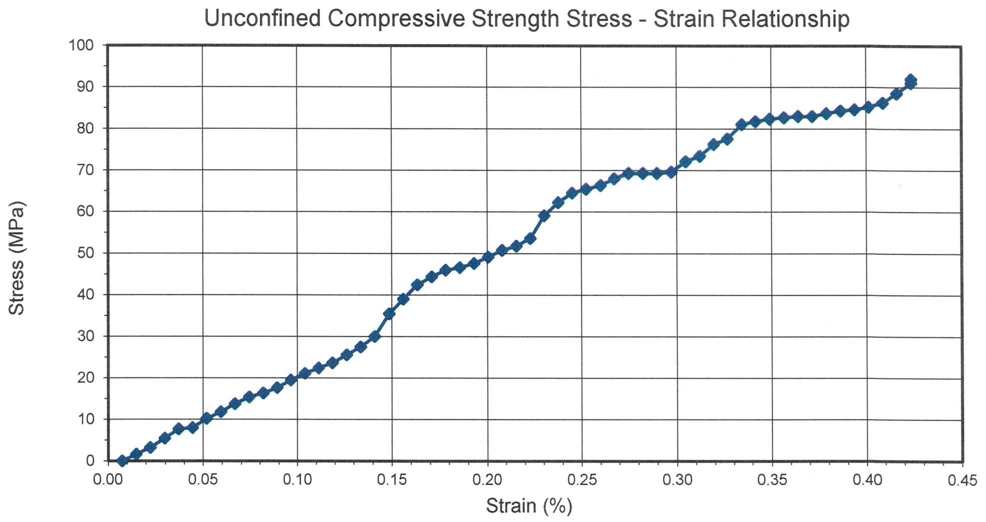
File No.: 24-027-01
Ref. No.: 24-27-1-19

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D. C09
Test Hole/Depth TH24-03, 53' 5.5" to 54' 1.5"
Date Cored: Aug 13/24
Date Received: Feb 7/25
Compression Machine Model: Soil Test CT-710

Submitted By: Client
Date Tested: Feb 14/25
Tested By: ENG-TECH (Kyle Zebiere)
Method: ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	23.0 °C
Average Length of Specimen:	134.50 mm	Average Diameter of Specimen:	63.25 mm
Load Rate: 0.12 kN/s	Maximum Load: 291.8 kN	Compressive Strength:	93 MPa

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

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UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

AECOM Canada Inc.
99 Commerce Drive
Winnipeg, Manitoba
R3P 1J9

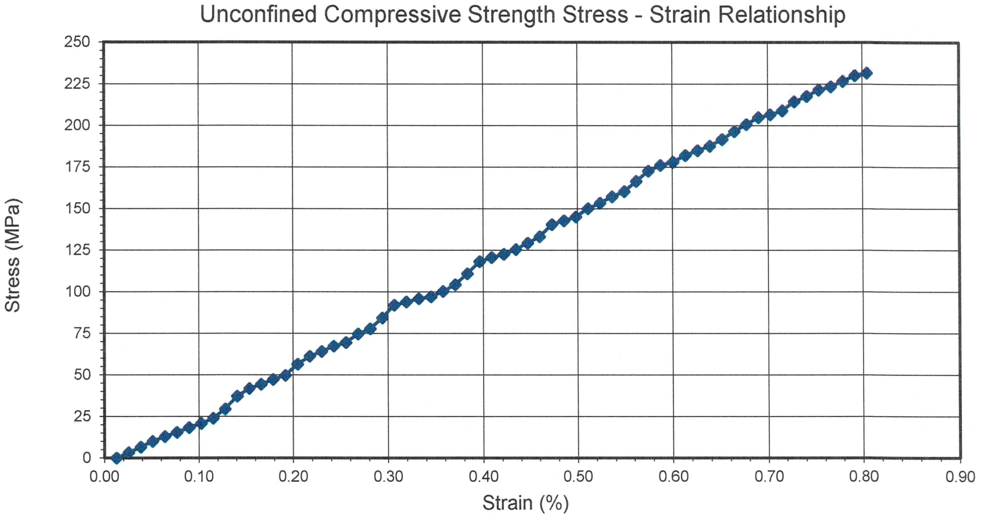
File No.: 24-027-01

Ref. No.: 24-27-1-20

Attention: Gene Acurin, E.I.T.

Project: PROJECT NO. 60728226, FORT GARY / ST. VITAL SIPHON RIVER CROSSING

Client I.D.: C10
Test Hole/Depth: TH24-03, 57' 3.5" to 58' 1.5"
Date Cored: Aug 13/24
Date Received: Feb 7/25
Compression Machine Model: Soil Test CT-710
Submitted By: Client
Date Tested: Feb 14/25
Tested By: ENG-TECH (Kyle Zebiere)
Method: ASTM D2938-95



Test Data

Specimen Moisture Condition:	As received moisture	Specimen Temperature:	23.0 °C
Average Length of Specimen:	156.50 mm	Average Diameter of Specimen:	63.10 mm
Load Rate: 0.12 kN/s	Maximum Load: 734.5 kN	Compressive Strength:	235 MPa

Reporting of these results constitutes a testing service only. Engineering interpretation or evaluation of the test results is provided only on written request.
*Denotes core Length/Diameter ratio not between 2.0 and 2.5.

Comments:

Deviation from test procedure: None

Email: AECOM Canada Inc. Contact Group

ENG-TECH Consulting Limited

Per

Darci Babisky, C.E.T.
Operations Manager - Laboratory
Ph: (204) 233-1694 Fx: (204) 235-1579

GEOMECHANICA

Geomechanica Inc.
Unit 14 – 1240 Speers Rd.
Oakville Ontario
Canada L6L 2X4

August 23, 2024

Gene Acurin
AECOM
99 Commerce Drive
Winnipeg, MB
Canada, R3P 0Y7

Re: CERCHAR Abrasivity Testing
(AECOM Project No. 60728226)

Dear Gene:

On July 17th, 2024 and August 16th, 2024 two (2) and three (3) HQ-sized core samples were received by Geomechanica Inc. via courier service. These samples were identified as being from AECOM project 60728226 (Replacement of FGSV Siphon Crossing the Red River Project). From these samples, a total of five (5) CERCHAR Abrasivity tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results are presented in the accompanying laboratory report and summary spreadsheet.

Sincerely,

Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:
Gene Acurin
AECOM
99 Commerce Drive
Winnipeg, MB
Canada, R3P 0Y7

Prepared by:
Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

August 23, 2024
Project number: 60728226

Abstract

This document summarizes the results of rock laboratory testing, including 5 CERCHAR Abrasivity tests. The CERCHAR Abrasivity Index (CAI) value(s) are presented herein.

In this document:

1 CERCHAR Abrasivity Tests

1.1 Overview

This section summarizes the results of CERCHAR abrasivity testing. Testing was performed using a Type-2 CERCHAR apparatus as shown in Figure 1a. The tips of the styluses were sharpened to a conical angle of 90° using the setup shown in Figure 1b. The styluses used to perform the tests are shown in Figure 1c-d (Rockwell hardness 55±1). A static force of 70 N was applied on top of the stylus by using a combination of weights. Details of the testing procedure are as follows:

1. The tips of the five styluses are sharpened using the grinding apparatus (Figure 1b).
2. The styluses are placed under a microscope (60x magnification) and three scaled photos (120° apart) are captured before the test is conducted to ensure the 90° point has been properly formed.
3. The test specimens are obtained by breaking core samples to expose a fresh fracture surface perpendicular to the core axis.
4. The specimen is secured in the cross-slide vise of the testing apparatus and the stylus is carefully lowered on to the surface of the rock.
5. A scratch measuring 10 mm in length is performed over a duration of 10 seconds. This process is repeated with all five styluses on undisturbed parts of the fracture surface (e.g., Figure 2a).
6. Lastly, the worn tips are re-examined under the microscope. From three scaled photos (120° apart), the wear flat, *d*, is measured (e.g., Figure 2c).

The length or the diameter of the wear flat, *d*, was measured from scaled microscope images using the image processing software Fiji (e.g., Figure 2b-c). The mean wear of the tip is calculated by taking the average *d* of all tests. The CERCHAR-Abrasivity-Index (CAI) of the sample is subsequently calculated by taking the mean wear and multiplying it by 10. The above testing procedure followed ASTM D7625.

1.2 Results

The results of CERCHAR abrasivity testing are provided in Table 1. Please note that additional specimen and testing details are available in the summary spreadsheet that accompanies this report.

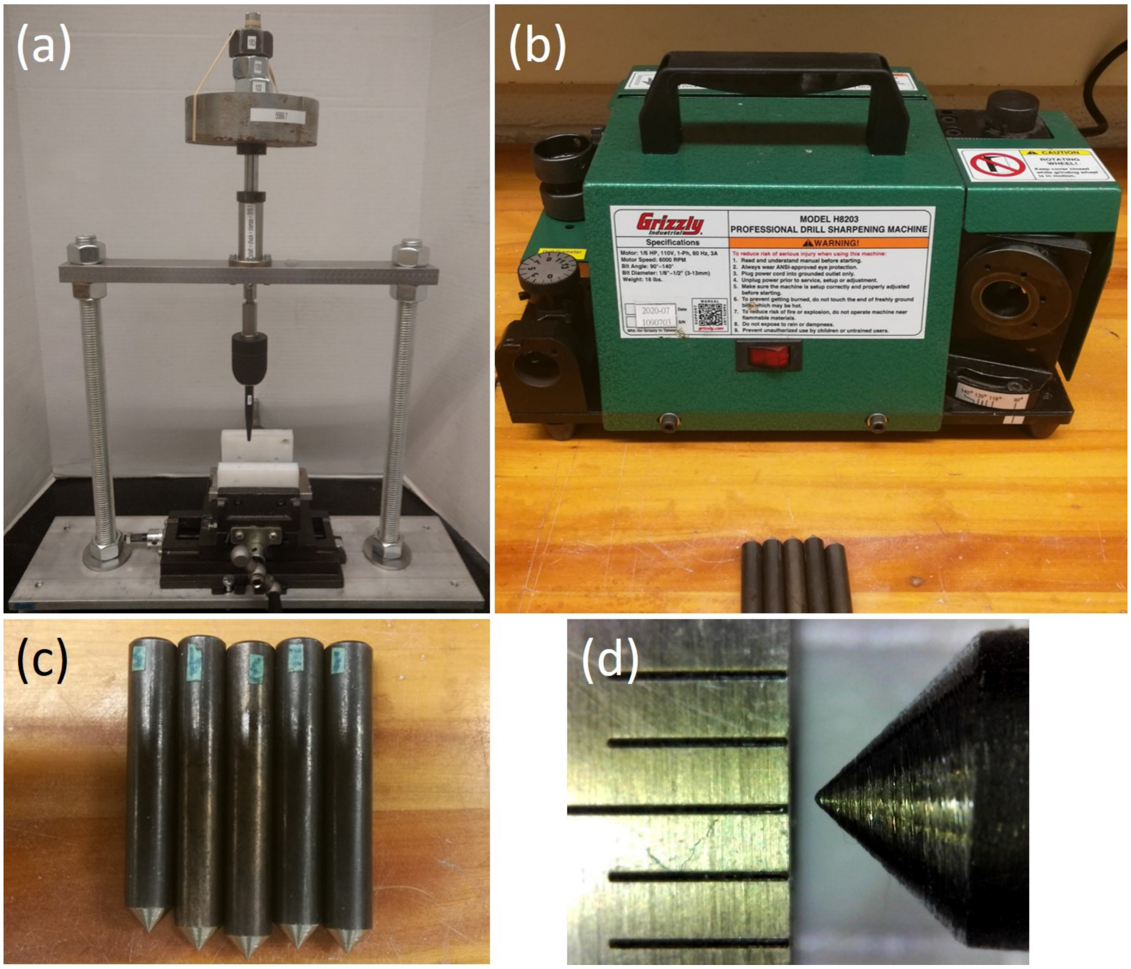
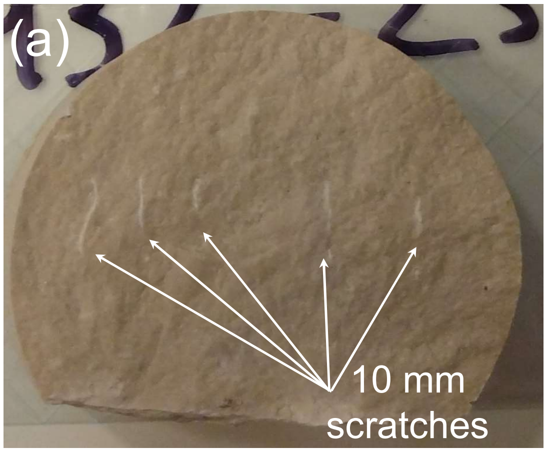
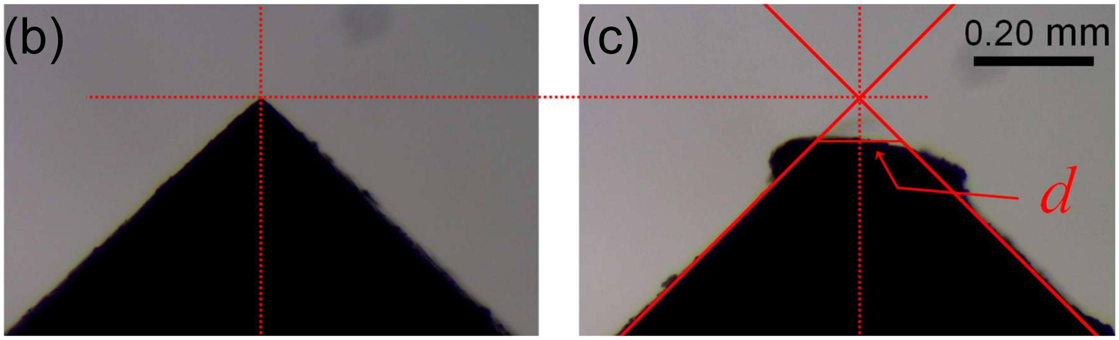


Figure 1: Photos showing (a) the CERCHAR apparatus, (b) tip sharpening setup, (c) the five styluses used to perform the test and (d) a microscope image of one of the stylus tips.



TH24-05, C23
23.43 - 23.61



TH24-05, C23
23.43 - 23.61
Stylus 1 pre-test @ 0°

TH24-05, C23
23.43 - 23.61
Stylus 1 post-test @ 0°

Figure 2: (a) Photograph showing an example of the five 10 mm scratches on a test specimen; (b) microscope image of select stylus prior to testing at the noted position; and (c) microscope image of the same stylus at the same position following testing with the wear flat, d, denoted.

Table 1: Summary of CERCHAR abrasivity test results.

Sample	Depth (m)	Test 1 Mean (mm)	Test 2 Mean (mm)	Test 3 Mean (mm)	Test 4 Mean (mm)	Test 5 Mean (mm)	Mean Wear (mm)	CAI	Lithology	ASTM Classification
TH24-01, C23	25.30 - 25.43	0.127	0.068	0.105	0.176	0.165	0.128	1.281	Lower Red River Formation - dolomitic mudstone, brecciated	Medium
TH24-05, C23	23.43 - 23.61	0.154	0.164	0.167	0.164	0.190	0.168	1.677	Lower Red River Formation - dolomitic mudstone, brecciated	Medium
TH24-03, C20	29.11 - 29.29	0.117	0.114	0.050	0.041	0.073	0.079	0.789	Lower Red River Formation - dolomitic mudstone, brecciated	Low
TH24-03, C21	31.13 - 31.32	0.059	0.055	0.029	0.034	0.034	0.042	0.423	Lower Red River Formation - dolomitic mudstone, brecciated	Very Low
TH24-03, C22	32.84 - 32.99	0.046	0.051	0.048	0.080	0.029	0.051	0.509	Lower Red River Formation - dolomitic mudstone, brecciated	Very Low

Rock Laboratory Testing Results

A report submitted to:
Gene Acurin
AECOM
99 Commerce Drive
Winnipeg, MB
Canada, R3P 0Y7

Prepared by:
Bryan Tatone, PhD, PEng
Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

February 20, 2025
Project number: 60728226

Abstract

This document summarizes the results of rock laboratory testing, including 2 CERCHAR Abrasivity tests. The CERCHAR Abrasivity Index (CAI) value(s) are presented herein.

In this document:

1 CERCHAR Abrasivity Tests

1.1 Overview

This section summarizes the results of CERCHAR abrasivity testing. Testing was performed using a Type-2 CERCHAR apparatus as shown in Figure 1a. The tips of the styluses were sharpened to a conical angle of 90° using the setup shown in Figure 1b. The styluses used to perform the tests are shown in Figure 1c-d (Rockwell hardness 55±1). A static force of 70 N was applied on top of the stylus by using a combination of weights. Details of the testing procedure are as follows:

1. The tips of the five styluses are sharpened using the grinding apparatus (Figure 1b).
2. The styluses are placed under a microscope (60x magnification) and three scaled photos (120° apart) are captured before the test is conducted to ensure the 90° point has been properly formed.
3. The test specimens are obtained by breaking core samples to expose a fresh fracture surface perpendicular to the core axis.
4. The specimen is secured in the cross-slide vise of the testing apparatus and the stylus is carefully lowered on to the surface of the rock.
5. A scratch measuring 10 mm in length is performed over a duration of 10 seconds. This process is repeated with all five styluses on undisturbed parts of the fracture surface (e.g., Figure 2a).
6. Lastly, the worn tips are re-examined under the microscope. From three scaled photos (120° apart), the wear flat, *d*, is measured (e.g., Figure 2c).

The length or the diameter of the wear flat, *d*, was measured from scaled microscope images using the image processing software Fiji (e.g., Figure 2b-c). The mean wear of the tip is calculated by taking the average *d* of all tests. The CERCHAR-Abrasivity-Index (CAI) of the sample is subsequently calculated by taking the mean wear and multiplying it by 10. The above testing procedure followed ASTM D7625.

1.2 Results

The results of CERCHAR abrasivity testing are provided in Table 1. Please note that additional specimen and testing details are available in the summary spreadsheet that accompanies this report.

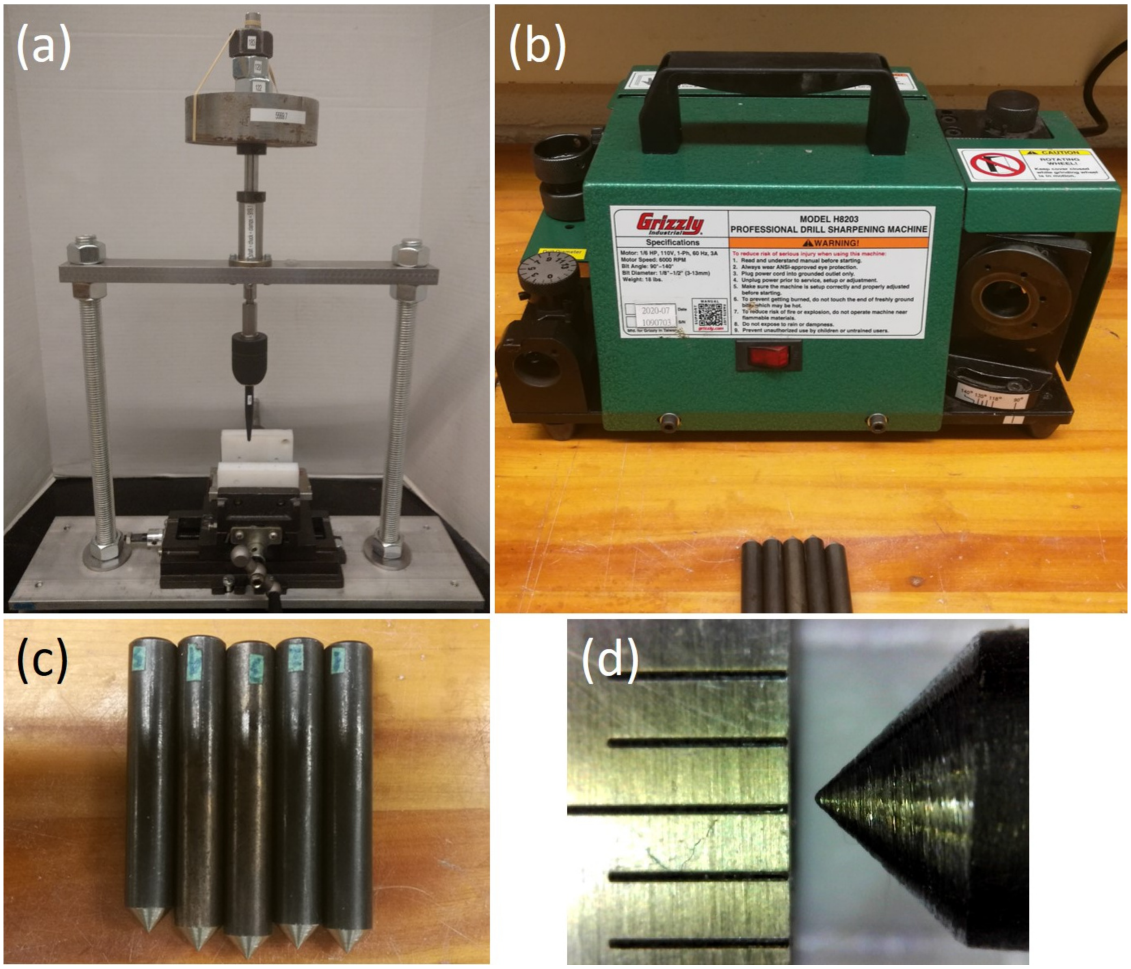
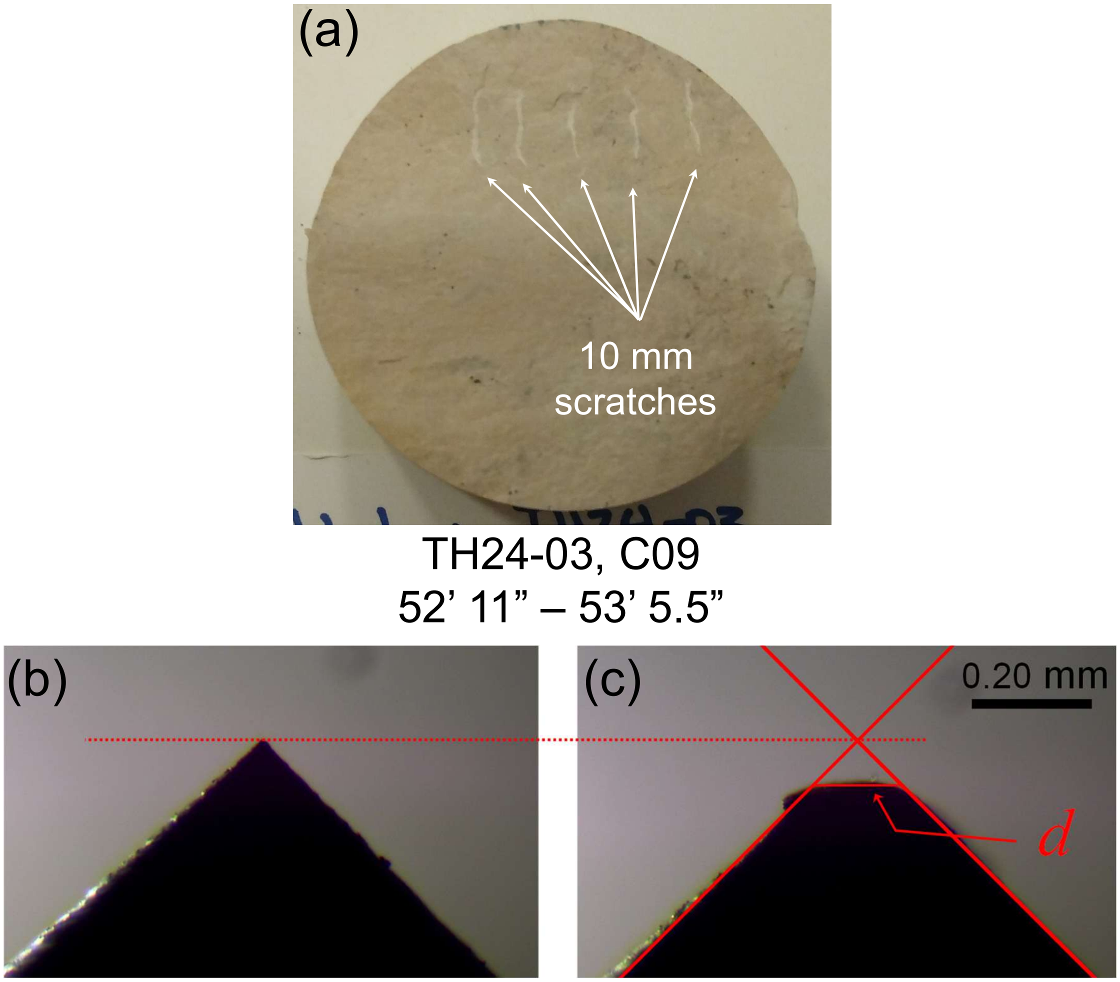


Figure 1: Photos showing (a) the CERCHAR apparatus, (b) tip sharpening setup, (c) the five styluses used to perform the test and (d) a microscope image of one of the stylus tips.



TH24-03, C09
52’ 11” – 53’ 5.5”

TH24-03, C09
52’ 11” – 53’ 5.5”
Stylus 1 pre-test @ 0°

TH24-03, C09
52’ 11” – 53’ 5.5”
Stylus 1 post-test @ 0°

Figure 2: (a) Photograph showing an example of the five 10 mm scratches on a test specimen; (b) microscope image of select stylus prior to testing at the noted position; and (c) microscope image of the same stylus at the same position following testing with the wear flat, d, denoted.

Table 1: Summary of CERCHAR abrasivity test results.

Sample	Depth (ft' in")	Test 1 Mean (mm)	Test 2 Mean (mm)	Test 3 Mean (mm)	Test 4 Mean (mm)	Test 5 Mean (mm)	Mean Wear (mm)	CAI	Lithology	ASTM Classification
TH24-03, C10	56'8" - 57'3.5"	0.157	0.152	0.140	0.151	0.159	0.152	1.517	Lower Red River Formation - dolomitic mudstone, brecciated	Medium
TH24-03, C09	52'11" - 53'5.5"	0.138	0.165	0.179	0.186	0.179	0.169	1.694	Lower Red River Formation - dolomitic mudstone, brecciated	Medium

Appendix 5

Seismic Hazard Values

Government of Canada Gouvernement du Canada

2020 National Building Code of Canada Seismic Hazard Tool

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 as prescribed in Article 1.1.3.1. of Division B of the NBC 2020.

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_E
Latitude (°)	49.822
Longitude (°)	-97.143

Please select one of the tabs below.

NBC 2020 Additional Values Plots API

Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2020. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.112	0.106	0.0546	0.0214	0.0043	0.00125	0.0677	0.054

The log-log interpolated 2%/50 year $S_a(4.0, X_E)$ value is : **0.0064**

▶ Tables for 5% and 10% in 50 year values

Download CSV

Go back to the seismic hazard calculator form

Date modified: 2021-04-06

Appendix 6

Technical Memorandum (AECOM, 2021)

AECOM
99 Commerce Drive 204 477 5381 tel
Winnipeg, MB, Canada R3P 0Y7 204 284 2040 fax
www.aecom.com

To: Armand Delaurier, Paul Bortoluzzi

Date: March 17, 2021

Project #: 60645745

From: Ryan Harras, B.Sc., P.Eng.
Elliott Drumright, PhD, P.E.

cc: Adam Braun (AECOM)

Technical Memorandum

Subject: **High Risk River Crossings – Phase 3 – Geotechnical Condition Assessment**

1. Introduction

1.1 General

The City of Winnipeg (City) has retained AECOM Canada Ltd (AECOM) to provide consulting services related to the condition assessment of High Risk Sewer and Water River Crossings (HRRC's) contained within the Phase 3 assessment program. As part of the stipulated condition assessment, geotechnical review was required at seven high risk crossing sites (Site 4 to Site 10).

The objective of the geotechnical assessment was to characterize the potential risk of slope instability and erosion as it relates to the serviceability of specific buried sewer and water systems at each of these crossing sites. Although commentary is provided on slope instabilities and erosion observed along the banks at each of the sites, the risk characterizations were based solely on existing bank features and conditions present that have the potential to engage the underlying utilities being studied. The findings of this assessment will assist the City in evaluating the probability of failure and managing these assets. The seven sites include: Fort Garry Bridge Siphon Crossings (Site 4), West Perimeter Bridge Force Main Crossing (Site 5), Dakota Feeder Main Crossing (Site 6A and Site 6B), Rouge Road Feeder Main Crossing (Site 7), West End (Omand's) Feeder Main Crossing (Site 8), West End (Truro) Feeder Main Crossing (Site 9), and the Haney-Moray Feeder Main Crossing (Site 10). It is understood that the remaining three high risk crossing sites (Site 1 to 3) are bridge-mounted, and therefore did not require a riverbank assessment as part of this scope of work.

The geotechnical component of the condition assessment included a review of available background information, followed by completion of a visual field inspection within a 30 m influence zone of each of the pipeline crossing sites. The findings and conclusions derived from the desktop review and visual field inspection were used to assign a Slope Condition Grade (SCG) and Erosion Condition Grade (ECG) related specifically to the risks the existing bank conditions pose to the utility lines, and assisted in identifying the sites that would need to be subjected to further geotechnical investigation and/or slope stability analyses.

This Technical Memorandum (TM) presents the findings of the geotechnical condition assessment completed for Site 4 to Site 10 and includes a summary of the results of background information review, visual field inspection, and assigned slope and erosion condition grades, as well as the results of the geotechnical investigations and slope stability analyses completed.

1.2 Background

The following geotechnical reports and studies were referenced in conjunction with this TM:

Site 4 (Fort Garry/St. Vital Interceptor Siphons – Red River)

- *AECOM Canada Ltd. (September 13, 2018) Technical Memorandum - High Risk River Crossings – Phase 2 – Geotechnical Assessment for Site 5 and 6. Ref. AECOM Project Number 60549028.*
- *AECOM Canada Ltd (December 12, 2013) Technical Memorandum - Preliminary Geotechnical Assessment Fort Garry Interceptor Sewer Crossing at the Red River.*
- *AECOM Canada Ltd (May 23, 2012) Technical Memorandum - Test hole adjacent to Interceptor, Fort Garry to St. Vital Interceptor, East Bank of Red River at Bishop Grandin Boulevard.*
- *Klohn Leonoff Consultants Ltd (April 5, 1976) Report on Sub-Soil Investigation - Fort Garry-St. Vital Corridor, Winnipeg, Manitoba.*

Site 5 (West Perimeter Bridge Force Main – Assiniboine River)

- *Geokwan Engineering Ltd. (October 25, 2000). Report on Sub-Soil Investigation. Proposed Perimeter West 600mm Outfall Sewer & 400mm Forcemain, Perimeter Hwy & Assiniboine River.*

Site 7 (Rouge Road Feeder Main – Sturgeon Creek)

- *KGS Group (October 2019). Report – Hamilton Avenue Bridge Outfalls - Preliminary Design Brief.*

Site 8 (West End Feeder Main – Omand's Creek)

- *UMA Engineering (August 5, 1987). Report - West End Feedermain Geotechnical Investigation.*
- *TREK Geotechnical (September 23, 2015). Report – Saskatchewan Avenue at Omand's Creek Bridge Replacement – Geotechnical Investigation.*

Site 9 (West End Feeder Main – Truro Creek)

- *UMA Engineering (August 5, 1987). Report - West End Feedermain Geotechnical Investigation.*

The following sources of information (varying in availability) were also referenced in review and evaluation of each HRRC site:

- As-built records.
- Aerial photography.
- Historic reports.
- Geological survey maps.
- Anecdotal information.

1.3 Bank Classification System

AECOM reviewed the City of Winnipeg's *Riverbank Stability Characterization Study (May 2000)* and assessed the banks at each HRRC site based on the basic classifications defined within the document. The bank classifications from this document are summarized as follows:

- *Failure Controlled Banks* – Are located in concave sections or outside bends of the river and are typically characterized by large deep-seated failures. Failures are typically within glaciolacustrine soils, and slopes generally achieve a quasi-stable configuration in the range of 6H:1V to 9H:1V
- *Erosion Controlled Banks* – Are located in convex sections or inside bends of the river and are typically characterized by localized shallow bank failures that occur due to excessive toe erosion. Failures are typically within alluvial soils, and slopes generally achieve a quasi-stable configuration in the range of 1H:1V to 3H:1V.

- *Transition Banks* – Are located in relatively straight river sections leading into convex/concave sections and are typically characterized by shallow and deep-seated failures. Failures may occur within alluvial and/or glaciolacustrine soils.
- *Altered Banks* – Consist of any of the above banks that have undergone remedial works to improve bank slope stability. These remedial works may include slope regrading, erosion protection (i.e. riprap armoring), shear keys, granular ribs, rock fill caissons, or retaining walls. Failures may still occur within these banks depending on the types and efficacy of the stabilization measures implemented.

Classification of the banks at each HRRC site were selected based on the geometry of the waterway, the results of the background information review, and the observations made during the visual field inspection.

1.4 Slope Condition Grade and Erosion Condition Grade System

AECOM implemented a SCG and ECG evaluation system at each of the sites. The SCG is directly analogous to the pipe's structural condition and is related to the structural stability of the overall slope that could engage the pipe. The ECG is analogous to the pipe's service ratings and is related to the toe erosion potential of the banks at each site and its potential ability to initiate or progress larger slope failures that may engage the pipe over time. The grading system is similar to the existing 5-point structural condition system identified by the Water Research Centre (WRC) and is summarized as follows:

- 1 = new asset or no defects present
- 2 = defects present, but short-term potential for further deterioration is low
- 3 = defects present, short-term potential for further deterioration is highly likely
- 4 = defects present of such a nature that a random event could initiate failure.
- 5 = defects present to the degree that failure has occurred or is incipient.

Sites with an SCG and/or ECG rating of 3 or above were considered for preliminary slope stability modelling and analyses that is discussed in subsequent sections.

2. Background Information Review

The following section summarize the results of the background information review at each HRRC crossing site.

2.1 Site 4: Fort Garry/St. Vital Interceptor Siphons (Red River)

- Asset: 700 mm and 800 mm HDPE Siphons.

Site 4 is located along the Red River at the Bishop Grandin Bridge crossing in south Winnipeg. The Red River crossing at Bishop Grandin Boulevard currently consists of two bridge structures with an under-bridge pedestrian crossing at both banks. An aerial location view of the site is shown in **Figure 2-1**.

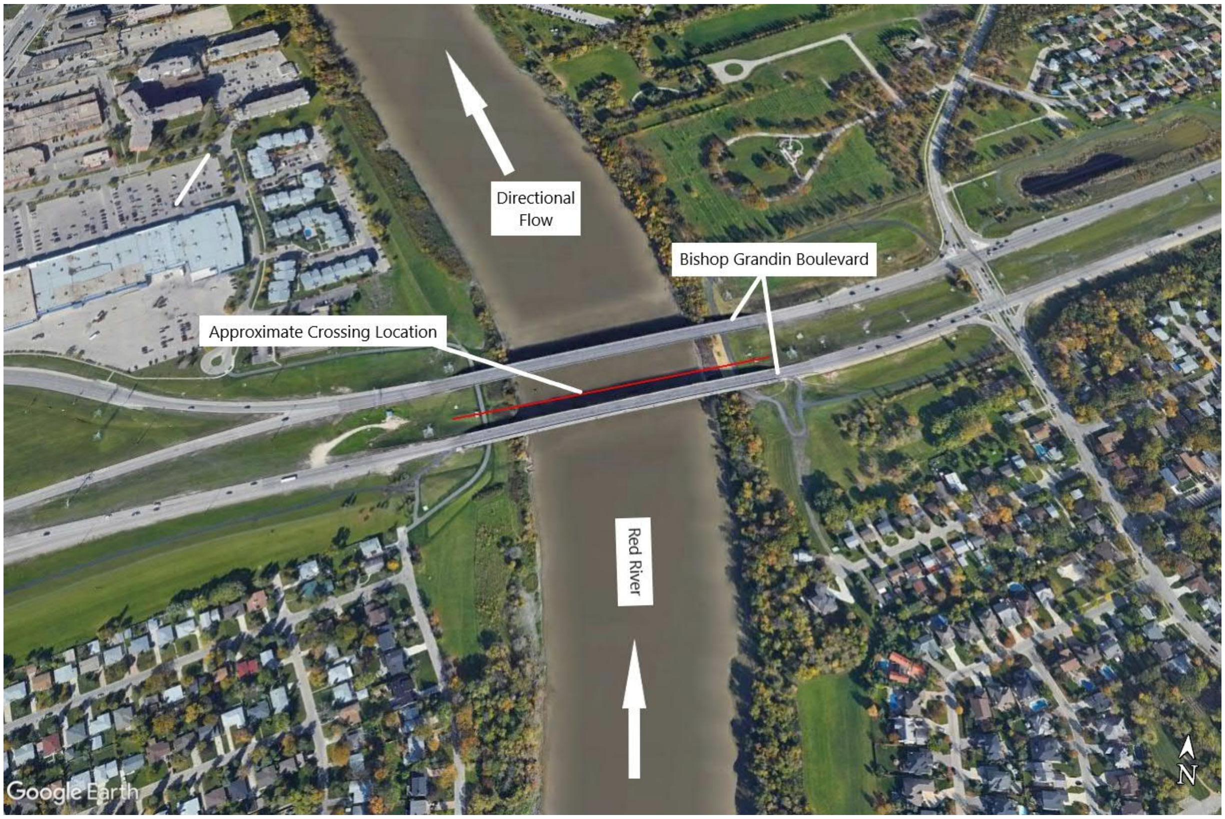


Figure 2-1 – Site 4 Location

The Red River flows north, with the crossing located near a gentle bend in the river. The west bank is on the inside of the bend (convex section) and the east bank is on the outside of the bend (concave section).

The Fort Garry/St. Vital interceptor siphon crossing is located within alluvial sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The alluvial soils that form the flood plain are comprised mainly of beds of clay, silt, sand, and gravel, which were deposited either directly on glacial till or on a layer of lacustrine clay. Existing test hole information indicates that the alluvial deposits are exposed over the full height of the subject riverbank throughout the study area.

The 700 mm and 800 mm buried siphons cross the river at approximate invert elevations ranging from 218.0 m to 219.5 m. The siphons rise significantly within the riverbank slopes to an invert elevation ranging from approximately 224.0 m to 226.0 m. The approximate locations of the siphons are shown on the as-built records attached in **Appendix A1**.

Klohn Leonoff Consultants Ltd. completed a subsurface geotechnical investigation at this site in 1975 and 1976 to determine subsurface ground and groundwater conditions at the site during design of the Bishop Grandin Bridges. An additional geotechnical investigation was completed by AECOM along the east bank in 2013 to provided subsurface information to assess the risk of slope instability with respect to the 800 mm siphon. The existing test hole logs and location plans that were available to AECOM at this site are attached in **Appendix B1**.

The geotechnical investigation completed by AECOM along the eastern riverbank slopes in 2013 concluded that slope conditions did not meet required factors of safety when assessed under short term conditions (i.e. rapid drawdown), which could potentially result in a slope failure engaging the existing 800 mm siphon within the eastern riverbank slope. The report recommended placement of stone riprap in-conjunction with slope regrading to mitigate the adverse effects of rapid drawdown on the bank stability. This work was completed in spring of 2014, along with repairs to the 800 mm interceptor at the eastern bank. Records of this work are included in **Appendix A1**.

2.2 Site 5: West Perimeter Force Main (Assiniboine River)

- Asset: 400 mm Steel Force Main

Site 5 is located along the Assiniboine River at the West Perimeter Highway Bridge crossing located near the west end of Winnipeg. The Assiniboine River crossing at the West Perimeter Highway currently consists of a single bridge structure with an under-bridge roadway at the north bank (Oxbow Bend Road). An aerial view of the site is shown in **Figure 2-2**.

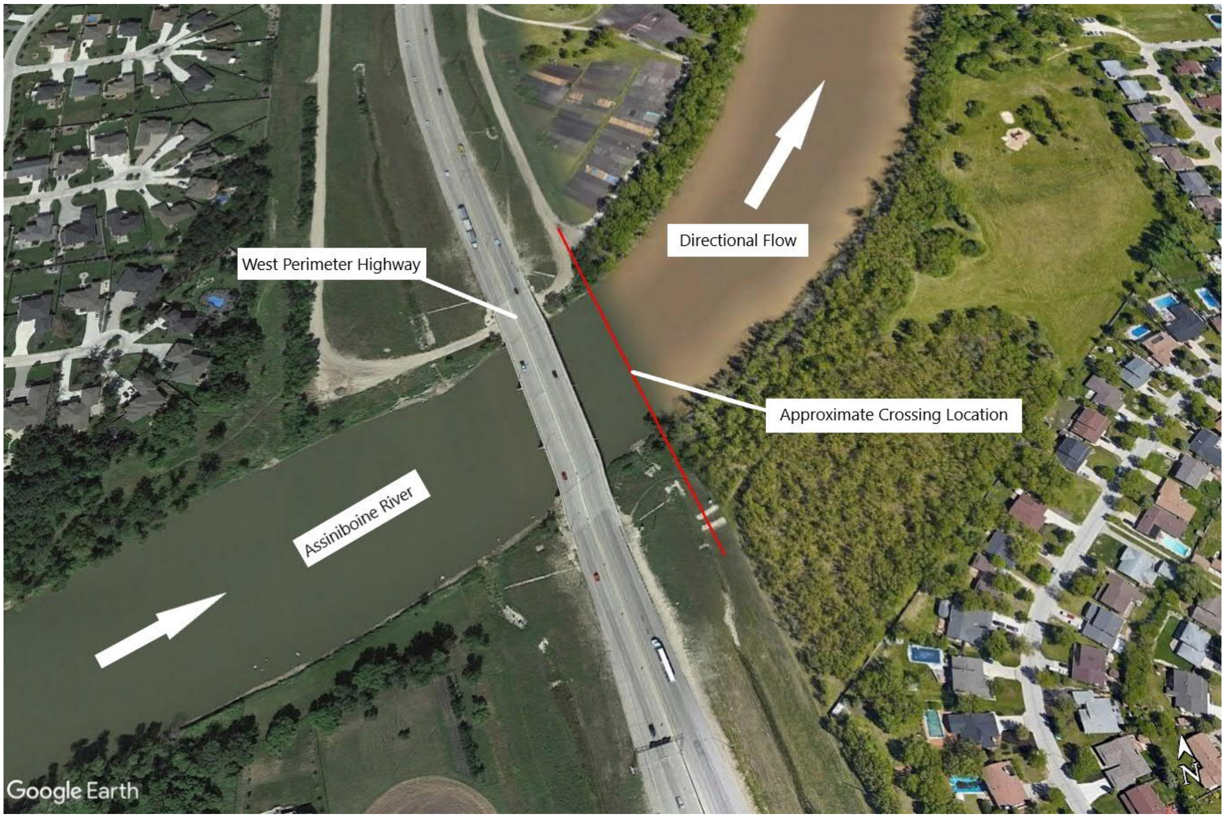


Figure 2-2 - Site 5 Location

The Assiniboine River flows approximately east, with the crossing located along a relatively straight stretch of the river, transitioning into a curve downstream of the crossing (with the south bank turning into an outside/concave bend, and the north bank turning into an inside/convex bend).

The West Perimeter Force Main crossing is located within an area of alluvial and glaciolacustrine sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The alluvial soils are typically comprised of beds of clay, silt, sand, and gravel, which were deposited either directly on glacial till or on a layer of lacustrine clay. The glaciolacustrine soils are comprised primarily of clays and silts, and were deposited from suspension within deep water of glacial Lake Agassiz. Existing test hole information indicates that alluvial and glaciolacustrine deposits were encountered within the study area.

The 400 mm buried force main crosses the river at an approximate invert elevation ranging from 226.6 m to 227.5 m. Within the north bank, the force main rises north of the riverbank slope crest to an approximate invert elevation of 230.5 m. Within the south bank, the force main rises gradually at a grade of approximately 1.4%. The approximate location of the force main is shown on the as-built records attached in **Appendix A2**.

Geokwan Engineering Ltd. completed a subsurface geotechnical investigation at this site in 2000 to determine subsurface ground and groundwater conditions at the site during design of the 400 mm steel force main. The existing test hole logs and location plan that were made available to AECOM are attached in **Appendix B2**.

2.3 Site 6: Dakota Feeder Main (Seine River and Navin Drain)

- Asset: 600 mm PCCP Feeder Main

Site 6 is located along the Seine River and Navin Drain, located north of Bishop Grandin Boulevard in south Winnipeg. The Navin Drain crossing location has been identified as “Site 6A”, while the Seine River crossing location has been identified as “Site 6B”. An aerial view of both crossings is shown in **Figure 2-3**.

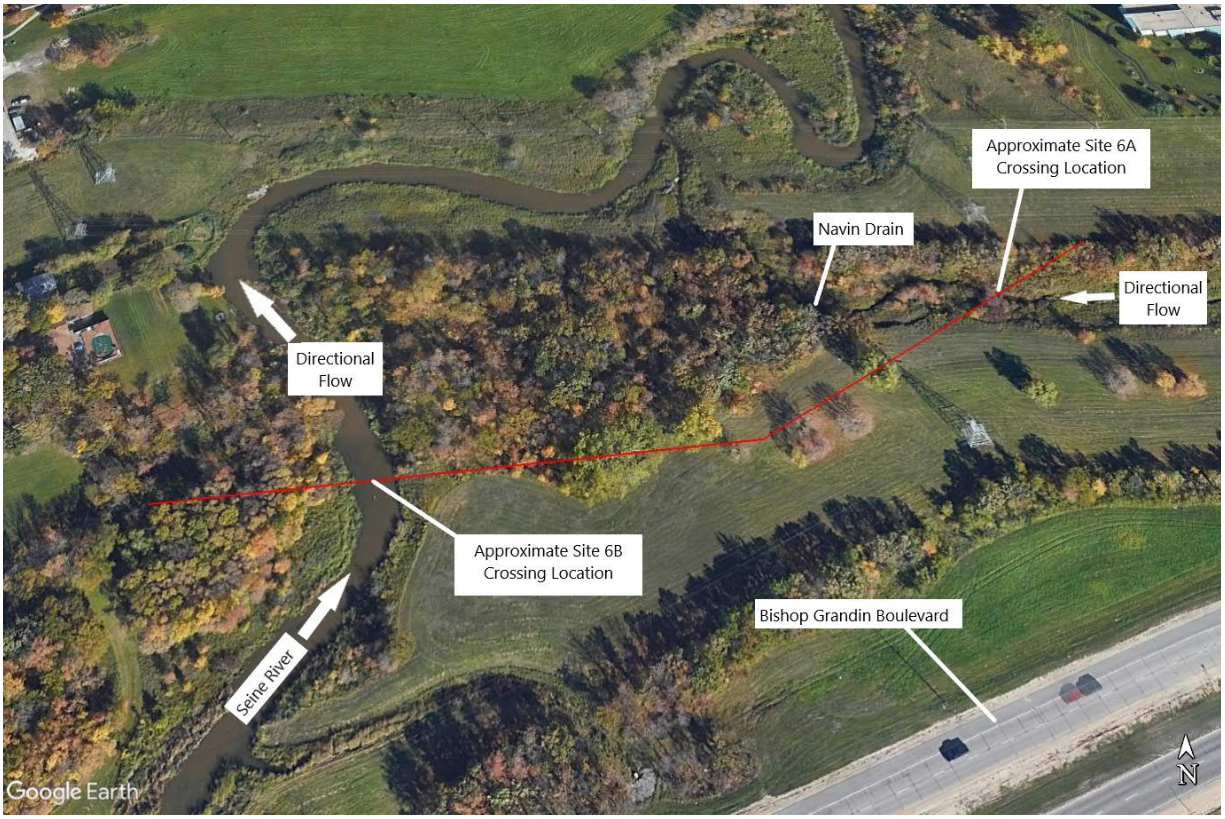


Figure 2-3 – Site 6 Location

The Navin Drain is a slightly meandering, man-made drainage channel that flows west and discharges into the Seine River. The Seine River flows generally north towards the Red River, with the Site 6B crossing located within a moderate bend in the river. The west bank is on the inside of the bend (convex section) and the east bank is on the outside of the bend (concave section).

Site 6A of the Dakota Feeder Main crosses the Navin Drain within glaciolacustrine sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). Glaciolacustrine soils are primarily comprised of clays and silts that were deposited from suspension within deep water of glacial Lake Agassiz.

Site 6B of the Dakota Feeder Main crosses the Seine River in an area of alluvial deposits as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The alluvial soils are comprised mainly of beds of clay, silt, sand, and gravel, which were deposited either directly on glacial till or on a layer of lacustrine clay.

The 600 mm feeder main crosses the Navin Drain and Seine River at approximate invert elevations of 224.0 m and 223.1 m, respectively. At points beyond the north and south bank slope crests of the Navin Drain (Site 6A), the feeder main rises to invert elevations ranging from 227.7 m to 228.0 m. Within the bank slopes of the Seine River (Site 6B), the feeder main rises to invert elevations ranging from 227.7 m to 228.0 m. The approximate location of the buried feeder main is shown on the as-built records attached in **Appendix A3**.

No existing geotechnical information at Site 6A and 6B was available for review.

2.4 Site 7: Rouge Road Feeder Main (Sturgeon Creek)

- Asset: 600 mm PCCP Feeder Main

Site 7 is located along Sturgeon Creek near the Hamilton Avenue Bridge in west Winnipeg. The Sturgeon Creek crossing at Hamilton Avenue currently consists of a single bridge structure with an under-bridge pedestrian crossing at both banks. An aerial view of the site is shown in **Figure 2-4**.

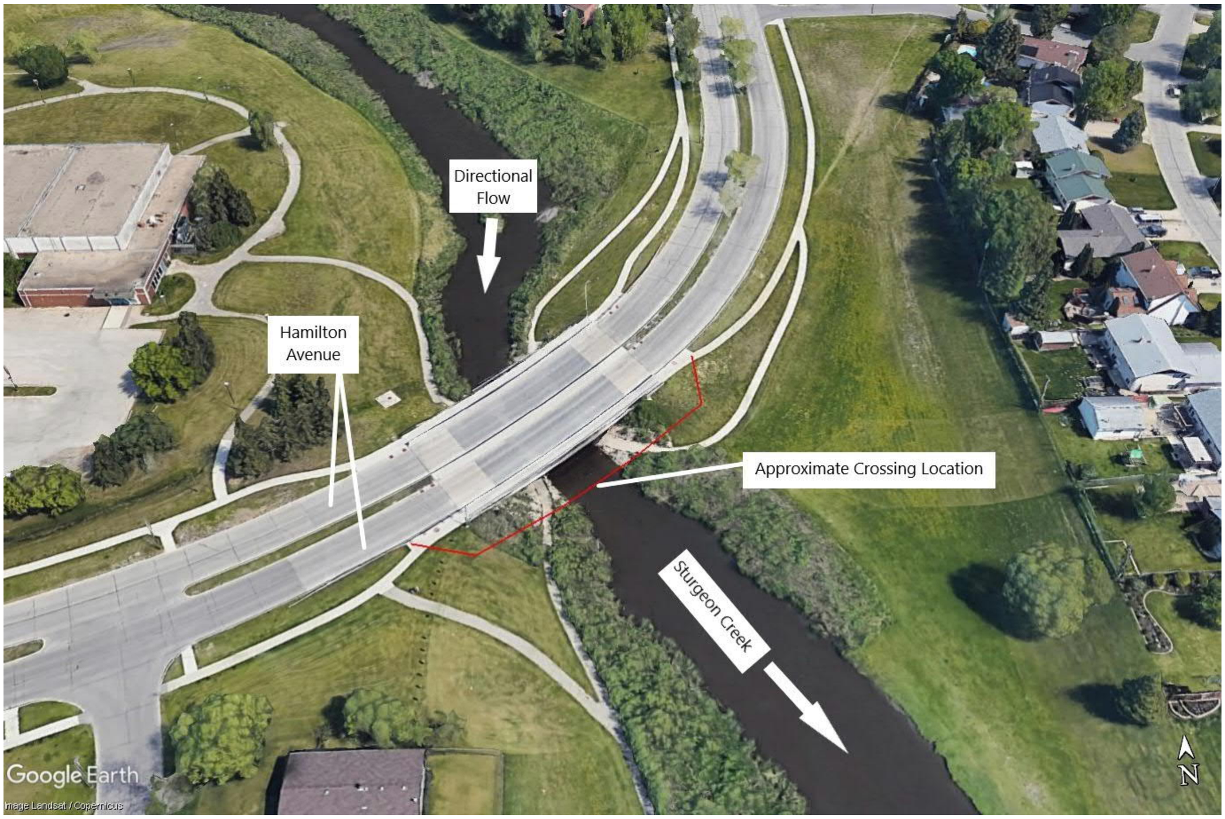


Figure 2-4 – Site 7 Location

Sturgeon Creek flows south towards the Assiniboine River, with the Site 7 crossing located within a straight portion of the creek immediately downstream of a creek bend.

The Rouge Road Feeder Main is located within an area of glaciolacustrine sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The glaciolacustrine soils are comprised primarily of clays and silts and were deposited from suspension within deep water of glacial Lake Agassiz. Existing test hole information north of the bridge site indicates that glaciolacustrine deposits were encountered in the vicinity of the study area.

The 600 mm feeder main crosses the creek at an approximate invert elevation of 228.9 m. Within the bank slopes, the feeder main rises within the slopes to an invert elevation of approximately 223.1 m at points just beyond the bank slope crests. The approximate location of the buried feeder main is shown on the as-built records attached in **Appendix A4**.

KGS Group completed a subsurface geotechnical investigation in the vicinity of this site in 2019 to determine subsurface ground and groundwater conditions at the site. The existing test hole logs and location plan that were made available to AECOM are attached in **Appendix B3**.

Information from the geotechnical investigation completed by KGS Group was used in developing slope stabilization measures on the north side of the bridge as part of the Hamilton Avenue Bridge Outfall Preliminary Design. The proposed works included regrading, placement of erosion protection, construction of a shear key, and filling of an observed sinkhole. This construction work is currently ongoing.

2.5 Site 8: West End Feeder Main (Omand's Creek)

- Asset: 900 mm PCCP Feeder Main

Site 8 is located along Omand's Creek at the Saskatchewan Avenue Bridge crossing. The Omand's Creek crossing currently consists of a relatively new roadway bridge structure (constructed in 2016) and two Canadian Pacific (CP) rail bridges upstream of it. An aerial view of the site is shown in **Figure 2-5**.

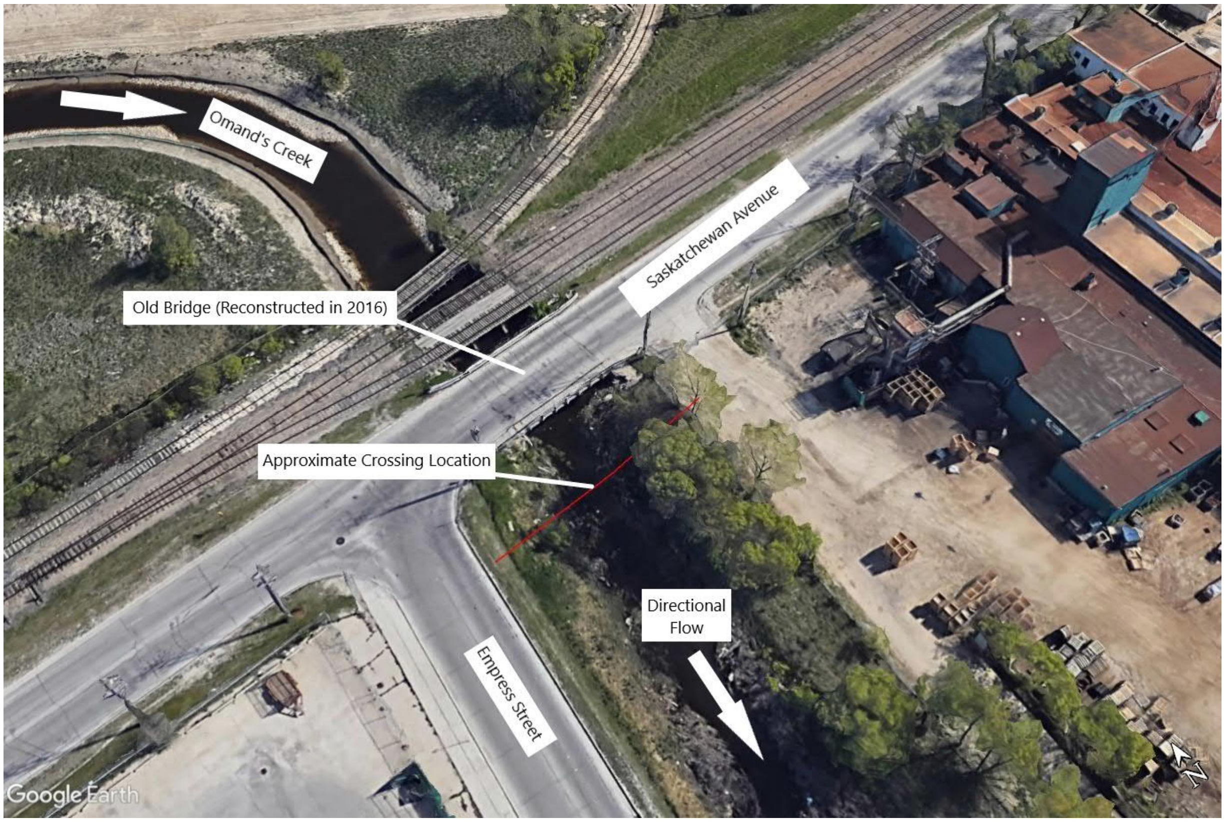


Figure 2-5 – Site 8 Location

Omand's Creek flows generally south towards the Assiniboine River, with the crossing located within a straight portion of the creek immediately downstream of a riprap-armoured creek bend.

The West End Feeder Main is located within an area of glaciolacustrine sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The glaciolacustrine soils are comprised primarily of clays and silts and were deposited from suspension within deep water of glacial Lake Agassiz. Existing test hole information indicates that glaciolacustrine deposits were encountered in the vicinity of the study area.

The 900 mm feeder main was installed within a hand-tunneled liner (backfilled with sand) in the vicinity of the crossing location, and crosses the creek at an approximate invert elevation of 228.5 m. At points beyond the east and west bank slope crests the feeder main rises to invert elevations ranging from 229.9 m to 230.9 m. The approximate location of the buried feeder main is shown on the as-built records attached in **Appendix A5**. However, it should be noted that the as-built information predates reconstruction of the Saskatchewan Avenue Bridge, and discrepancies were noted between information provided in the as-built drawings and observed site conditions at the crossing location with respect to bank geometry and riprap presence.

UMA Engineering Ltd. completed a subsurface geotechnical investigation along the feeder main alignment in the vicinity of this site in 1986 to determine subsurface ground and groundwater conditions during design of the West End Feeder Main. An additional geotechnical investigation was completed by TREK Geotechnical Inc. in 2015 to provide subsurface information for the purpose of design and reconstruction of the Saskatchewan Avenue Bridge. The existing test hole logs and location plans that were made available to AECOM have been attached in **Appendix B4**.

The 1986 geotechnical investigation by UMA included slope stability analyses at the Omand's Creek crossing, which indicated marginal factors of safety for shallow slip surfaces (consistent with observed over steepened bank conditions and observable instabilities), and adequate factors of safety for slip surfaces intersecting the proposed feeder main. The geotechnical investigation completed by TREK at the Saskatchewan Avenue Bridge site in 2015 also included slope stability analyses related to the proposed bridge infrastructure and existing feeder main. The results of the analysis indicated marginal factors of safety for the existing bank geometries and adequate factors of safety for slip surfaces intersecting the existing feeder main. As part of the bridge construction works, regrading and riprap armouring of the slopes to the south of the proposed bridge structure were proposed, and factors of safety for slip surfaces intersecting the existing feeder main were further improved. Construction of the proposed new bridge including regrading and riprap armouring to the south of the bridge was completed in 2016.

2.6 Site 9: West End Feeder Main (Truro Creek)

- Asset: 900 mm PCCP Feeder Main

Site 9 is located along Truro Creek southwest of the Silver Avenue Pathway pedestrian bridge, and east of the Assiniboine Golf course. An aerial view of the site is shown in **Figure 2-6**.

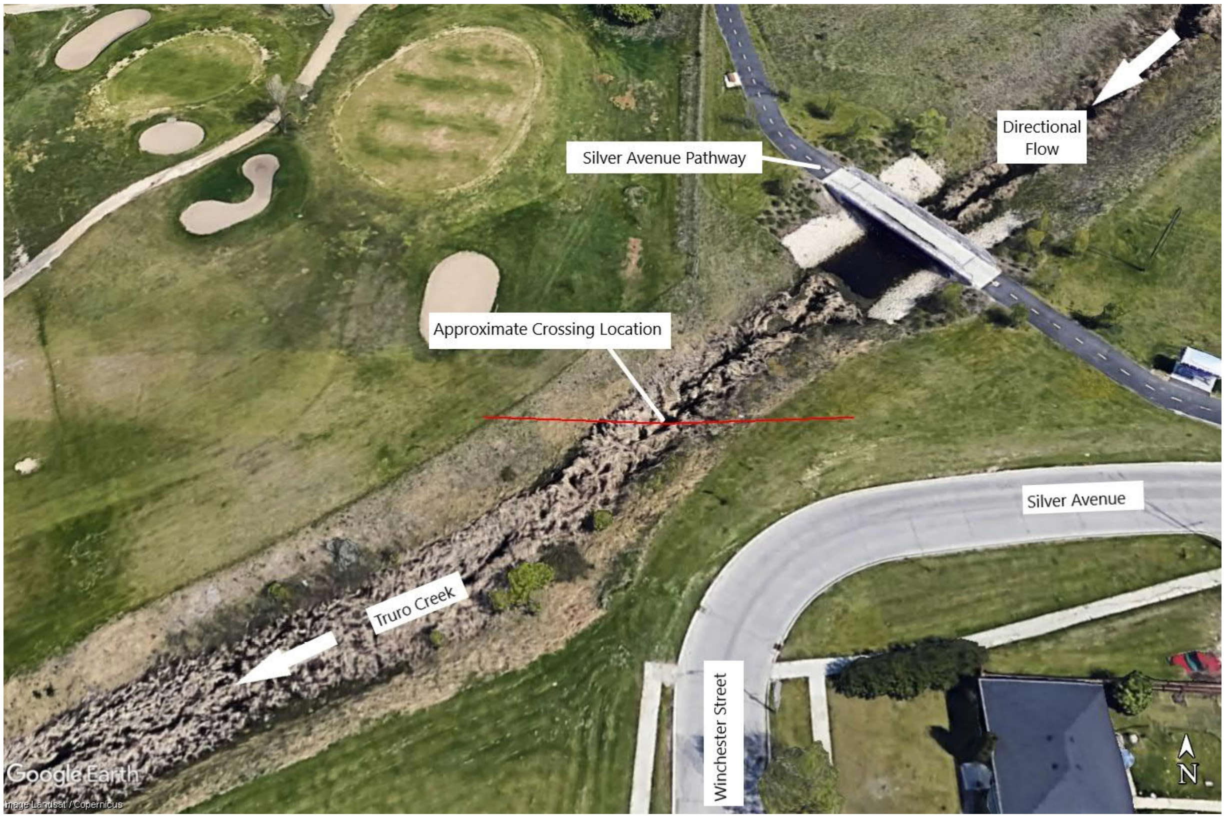


Figure 2-6 – Site 9 Location

Truro Creek flows south towards the Assiniboine River, with the pipeline crossing the creek on a skew within a straight portion of the creek immediately upstream of a gentle bend in the creek.

The West End Feeder Main is located within an area of glaciolacustrine sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The glaciolacustrine soils are comprised primarily of clays and silts and were deposited from suspension within deep water of glacial Lake Agassiz. Existing test hole information north of the bridge site indicates that glaciolacustrine deposits were encountered in the vicinity of the study area. The 900 mm feeder main crosses the creek at an approximate invert elevation of 227.7 m. Within the bank slopes, the feeder main rises within the slopes to an invert elevation ranging from approximately 231.1 m to 231.3 m at points near the bank slope crests. The approximate location of the buried feeder main is shown on the as-built records attached in **Appendix A6**.

UMA Engineering Ltd. completed a subsurface geotechnical investigation along the proposed feeder main in the vicinity of this site in 1986 to determine subsurface ground and groundwater conditions during design. The existing test hole logs and location plan that were made available to AECOM at this site have been attached in **Appendix B5**.

The geotechnical investigation by UMA included slope stability analyses at the Truro Creek crossing which indicated factors of safety for shallow slip surfaces and slip surfaces intersecting the pipe that were slightly below design factors of safety. Recommendations were made for the slopes to be regraded upon completion of construction.

2.7 Site 10: Haney-Moray Feeder Main (Assiniboine River)

- Asset: 450 CPP Feeder Main

Site 10 is located along the Assiniboine River at the William R. Clement Parkway Bridge crossing. The crossing currently consists of two bridge structures with an under-bridge pedestrian crossing at both banks. An aerial view of the site is shown in **Figure 2-7**.

Figure 2-7 – Site 10 Location

The Assiniboine River flows east, with the crossing located within a gentle bend in the river. The north bank is on the outside of the bend (concave section) and the south bank is on the inside of the bend (convex section).

The Haney-Moray Feeder Main crossing is located within an area of alluvial sediments as per the Surficial Geology map of Winnipeg (*MGS Geoscientific Map 2003-7*). The alluvial soils are typically comprised of beds of clay, silt, sand, and gravel, which were deposited either directly on glacial till or on a layer of lacustrine clay.

The 450 mm feeder main crosses the river at an approximate invert elevation ranging from 225.1 m to 225.2 m. Within the bank slopes, the feeder main rises to an approximate invert elevation ranging from 226.5 m to 229.2 m. The approximate locations of the buried siphons are shown on the as-built records attached in **Appendix A7**. However, it should be noted that the as-built information predates construction of the William R. Clement Parkway Bridge, and discrepancies were noted between information provided in the as-built drawings and observed site conditions at the crossing location with respect to slope regrading and riprap armouring near the river edge.

No existing geotechnical information was available for review at this site.

2.8 Site Surveys

Topographic surveys were not included as part of the geotechnical field program, and as such, all subsequent geotechnical analyses have been based on previous topographic surveys, LIDAR information (City of Winnipeg 2011 Data Set) and previous studies conducted within the crossing areas. The positions of known sewer and water systems have been inferred from as-built records and incorporated into the geotechnical analysis.

3. Visual Field Inspection

3.1 General

Field inspection of Sites 4 through 10 was undertaken between November 17 and 18, 2020 by AECOM geotechnical personnel to document and photograph existing site conditions as they related to the river/creek bank slopes (i.e. instabilities, tension cracking, erosion scarps, etc.), existing structures (i.e. detected displacement, detected damage, etc.), and vegetation (i.e. type of vegetation, density of vegetation, displacement of vegetation, etc.).

Results of the background information review and the visual field inspection at each site were used to assign appropriate SCG and ECG values and determine the need for subsequent geotechnical investigation, laboratory testing, instrumentation monitoring and slope stability analysis. Sites with an SCG and/or ECG greater than or equal to 3 were flagged for preliminary slope stability analysis.

Photographs taken throughout the course of the field inspection visits are presented as **Appendix C**. A summary of the observations noted during the site reconnaissance and the SCG and ECG ratings selected for each site are presented in **Appendix D**.

3.2 Site 4: Fort Garry/St. Vital Interceptor Siphons (Red River)

General observations of the west bank during the field inspection indicated minor erosion scarps, as well as a scarp near the crest of the riverbank likely resulting from shallow failures within over steepened portions of the riverbank. There was no evidence of deep-seated or rotational failures along this bank. The presence of localized riprap near the toe of the riverbank around the crossing alignment indicates that the west bank would be appropriately classified as an altered bank.

General observations of the east bank during the field inspection indicated minor erosion above the riprap armoured area near the bank toe. The riprap in this area was placed as part of the 2013 slope stabilization measures, and as a result, the east bank would be most appropriately classified as an altered bank.

3.2.1 Riverbank Slope Observations

3.2.1.1 Western Riverbank

- West of the asphalt sidewalk (orientated north to south), the ground surface between the Fort Garry bridges falls gently east towards the bridge abutments. The slope profile changes at a point almost in line with the bridge abutments within the study area, sloping more sharply towards the sidewalk, and then becomes more gradual between the sidewalk and the riverbank crest.
- The crest of the riverbank slope is approximately 20 m east of the sidewalk edge, and the surface of the riverbank was visible for approximately 10 m horizontally until intercepting the water's edge further downslope. The upper portion of the exposed riverbank slope was generally covered in shrubs and bushes, while the lower portion had riprap placed in close proximity to the crossing locations and exposed alluvial soils elsewhere.

- The profile of the riverbank slope from the crest down to the water's edge was estimated to range between 2H:1V to 3H:1V.
- Stone riprap was present around the two bridge abutments and was also observed to be present approximately 3 to 5 m on either side of the siphon crossing alignments (total length of armoring around crossing was between 6 and 10 m). The riprap was generally large (greater than 600 mm) and in places appeared to be moving down slope towards the river. Some loss of riprap around the bridge abutments has exposed the underlying alluvial soils.
- Erosion has resulted in gullying and material loss in and around the bridge abutment riprap as a consequence of surface water flow from the culverts west of the riverbank. Gullies measuring a depth of up to 400 mm were recorded.
- Erosion scarps were noted at the river edge and at various distances from the river edge, indicative of erosion occurring at different river levels. These erosional scarps were typically 100 mm to 150 mm in vertical height, and present in areas that were not amoured with riprap.
- Erosion horizontally into the riverbank was observed in localized areas that were not amoured with riprap.
- A vertical scarp approximately 300 mm in height was observed in a localized section of the riverbank near the crest. This scarp suggested the presence of shallow slope failures in areas where the riverbank was over steepened beyond 2H:1V.
- No evidence of deep-seated slope instabilities was noted within the riverbank slope.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.2.1.2 Eastern Riverbank

- East of the asphalt sidewalk (orientated north to south), the ground surface between the Fort Garry bridges gently falls west towards the bridge abutments. The slope profile changes at a point almost in line with the bridge abutments within the study area, sloping more sharply towards the sidewalk pavement and riverbank crest
- The crest of the riverbank slope was approximately 10 meters west of the sidewalk edge, and the surface of the riverbank was visible for approximately 15 m horizontally until intercepting the water's edge further downslope. The upper portion of the exposed riverbank slope was generally covered in shrubs and bushes, while the lower portion had riprap placed for the full length of riverbank between the two bridge structures.
- The profile of the riverbank slope from the crest down to the water edge was estimated to range between 3H:1V to 4H:1V.
- Stone riprap placed around the bridge piers was not noted to extend beyond the limits of the bridge by more than a few meters. Considerably less riprap was observed around the northern bridge pier as compared to the south bridge pier. Some loss of riprap around the bridge piers has exposed the underlying alluvial soils.
- Stone riprap was present along the lower portion of the riverbank for the full length between the bridge structures. The riprap was generally large (greater than 600 mm) and partially buried below fine-grained soils.
- Erosion scarps were noted at various distances from the river edge, indicative of erosion occurring at different river levels. These erosional scarps were typically 100 mm in vertical height, and present in areas above the riprap armoring.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- No evidence of animal burrows or infestations were noted within the riverbank slope. However, animal burrows were frequently observed within the ground surface to the east of the sidewalk.

3.2.2 Existing Structures

3.2.2.1 Western Riverbank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structures (2) - including superstructure and substructures (abutments and piers)
    - Lift station (and associated valve chambers)
    - Monitoring station(s)
    - Drainage Culverts
    - Hydro Tower
    - Asphalt Sidewalk
- The existing sidewalk pavement showed signs of distress in some locations within the study area adjacent to the riverbank crest. Cracks within the asphalt surface were orientated in a north south direction running parallel to the riverbank crest.
- All other structures outlined above visually appeared in good condition.

3.2.2.2 Eastern Riverbank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structures (2) - including superstructure and substructures (abutments and piers)
    - Valve Chamber
    - Drainage Culverts
    - Hydro Tower
    - Asphalt Sidewalk
    - Geotechnical Instrument - Groundwater Monitoring Well
- The ground immediately surrounding the hydro tower appeared to be undermined due to a combination of animal burrows and over steepened side slopes. The foundation fill used to elevate the towers was sloped at an approximate profile of 2H:1V and showed signs of slope bulging near the toe. The towers are somewhat removed from the riverbank slopes in the immediate study area and are deemed not to have any direct impact upon riverbank stability.
- The existing sidewalk pavement showed signs of distress in some locations within the study area adjacent to the riverbank crest. Cracks within the asphalt surface were orientated in a north south direction running parallel to the riverbank crest.
- All other structures outlined above visually appeared in good condition.

3.2.3 Vegetation

3.2.3.1 Western Riverbank

- West of the sidewalk observed vegetation consisted of maintained grass lawn.
- East of the sidewalk and west of the riverbank crest the vegetation primarily consisted of shrubs and bushes.
- Several large mature trees were identified in clusters near the riverbank crest.
- The upper portion of the riverbank slope was covered with shrubs and brush.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.2.3.2 Eastern Riverbank

- East of the sidewalk observed vegetation consisted of maintained grass lawn.
- West of the sidewalk the vegetation primarily consisted of shrubs and bushes.
- Some trees were identified in clusters near the riverbank crest.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.2.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-1: Summary of SCG and ECG Values (Site 4)

Bank	SCG	ECG	Comments
West	3	2	Evidence of slope instabilities and erosion indicated need for further analysis. Slope stability analysis completed at this site and results presented in Section 5.
East	1	2	No defects observed with slope condition. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.3 Site 5: West Perimeter Force Main (Assiniboine River)

General observations of the north bank during the field inspection indicated the presence of scarps of varying height mid-way up the riverbank, potentially due to a combination of riverbank erosion and shallow-seated slope instabilities driven by the erosion. There was no evidence of deep-seated or rotational failures along this bank. Riprap was not present within the crossing alignment but was observed around adjacent drainage infrastructure within the study area. Based on the background information review and results of the visual field inspection, the north bank would be appropriately classified as a transition bank.

General observations of the south bank during the field inspection indicated the presence of scarps of varying height near the river edge, potentially due to riverbank erosion. Riprap was observed near the toe of the riverbank slightly west of the approximate crossing alignment and appears to effectively prevent bank erosion due to surficial drainage discharge from two existing large-diameter CSP culverts. The gradually sloping nature of the area and the drainage features installed suggest that regrading work was likely done during construction of the Perimeter Highway bridge. Therefore, the south bank would be appropriately classified as an altered bank.

3.3.1 Riverbank Slope Observations

3.3.1.1 Northern Riverbank

- The ground surface along Oxbow Bend Road (east of the Perimeter Highway bridge) gently falls south towards the river.
- Within the eastern portion of the study area, the slope profile changes at the riverbank crest near the tree line, sloping more sharply towards the river at approximately 2.5H:1V before flattening out in advance of an observed scarp. The riverbank from the scarp to the water edge is at an approximate slope of 3H:1V. Within the western portion of the study area, the slope profiles changes at the riverbank crest located immediately south of the southern edge of Oxbow Bend Road, sloping more sharply down towards the river at approximately 3H:1V to 4H:1V.
- The upper portion of the exposed riverbank slope was generally covered in shrubs and bushes, while the lower portion had a thinner brush cover and some exposed alluvial soils.
- Stone riprap was observed around the bridge abutment and pier, within the discharge path of a concrete culvert crossing below Oxbow Bend Road near the bridge, and within the discharge path of a CSP culvert. The riprap was generally large (300 mm to 600 mm) and showed some displacement down the slope towards the river.
- Erosion has resulted in some gullying and material loss within the CSP culvert discharge path as a consequence of surface water flow.
- Scarps were noted approximately 2 to 3 m away from the river edge, indicative of potential erosion and/or shallow slope instabilities. These scarps typically ranged in vertical height from 300 mm to

900 mm within the study area (smaller to the west, larger to the east), but were not present in areas amoured with riprap.
- No evidence of deep-seated slope instabilities was noted within the riverbank slope.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.3.1.2 Southern Riverbank

- The ground surface between the eastern tree line and the Perimeter Highway bridge to the west slope steeply downwards into a riprap lined drainage channel. The steep slopes leading down to the drainage channel had large diameter rock drains installed within them. From the drainage channel, the site gradually falls north towards the river.
- The slope profile changes approximately 20 m south of the riverbank crest, sloping more sharply towards the river at approximately 5H:1V before flattening out in advance of an observed scarp. The riverbank from the scarp to the water edge is at an approximate slope of 2H:1V to 2.5H:1V.
- The upper portion of the exposed riverbank slope was generally covered in shrubs and bushes, while the lower portion had exposed alluvial or glaciolacustrine soils.
- Stone riprap was observed around the bridge abutment and pier, and within the discharge path of the two large diameter CSP culverts and was generally large (600 mm). Sporadic displaced riprap was also observed between the scarp and the river edge west of the crossing location within the flow path of the CSP culverts.
- Scarps were noted approximately 1 to 2 m away from the river edge, indicative of erosion. These scarps typically ranged in vertical height from 300 mm to 600 mm within the study area but were not present in areas amoured with riprap.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.3.2 Existing Structures

3.3.2.1 Northern Riverbank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structure - including superstructure and substructures (abutments and piers)
    - Drainage Culverts – Concrete and CSP
    - Concrete Drainage Flume
    - Granular Roadway – Oxbow Bend Road
    - Jersey Barrier at Road Edge
    - Traffic Signage
- One of the traffic signs was leaning towards the river, potentially due to slope movement, or more likely being struck by something (since sign directly beside it was vertical).
- All other structures outlined above visually appeared in good condition.

3.3.2.2 Southern Riverbank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structures - including superstructure and substructures (abutments and piers)
    - Drainage Culverts - CSP
    - Lift Station
- South end of eastern CSP was observed to have a slight bend near its crest.
- All other structures outlined above visually appeared in good condition.

3.3.3 Vegetation

3.3.3.1 Northern Riverbank

- Mowed lawn west of Oxbow Bend Road (bridge abutment)
- Within the eastern portion of the study area the riverbank slopes were heavily vegetated with large mature trees and dense brush. Between the observed scarp and river's edge, the vegetation generally consisted of sparse brush.
- Within the western portion of the study area the riverbank slopes were primarily vegetated with brush and shrubs, becoming sparse between the observed scarp and river's edge. Multiple large mature trees were identified in clusters within the upper half of the riverbank.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.3.3.2 Southern Riverbank

- Within the eastern portion of the study area the riverbank slopes were heavily vegetated with large mature trees and dense brush. Between the observed scarp and river edge, vegetation was typically not observed.
- Within the western portion of the study area the riverbank slopes were primarily vegetated with brush and shrubs. Between the observed scarp and river edge, the vegetation generally consisted of sparse brush. A few large mature tree clusters were observed within the gradually sloping portion of the riverbank.
- A downed tree was observed in the vicinity of the crossing location, appearing to have been uprooted by progressive riverbank erosion.
- Other than the single downed tree, there was no widespread indication of significant vegetation movement resulting from slope instability within the study area.

3.3.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-2: Summary of SCG and ECG Values (Site 5)

Bank	SCG	ECG	Comments
North	2	2	Evidence of minor slope instabilities and erosion. Asset installed within glacial till at crossing. Short-term potential for further deterioration of asset due to slope instability and erosion is low.
South	2	2	Evidence of minor slope instabilities and erosion. Asset installed within glacial till at crossing. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.4 Site 6A: Dakota Feeder Main (Navin Drain)

During background information review, the north and south riverbanks of the Navin Drain were classified as altered banks given that the drain is not a naturally occurring waterway, but rather a constructed one.

General observations made at the north bank during the visual field inspection indicated the presence of over steepened slopes, scarps near the bank crest indicative of shallow or potentially deep slope instabilities, shallow slope instabilities near the bank toe, and erosion scarps at the toe of the bank. Identification of the slope instability mechanisms (i.e. tension cracks, bulging, scarps, etc.) could not be identified in detail due to the dense brush cover at the time of the inspection. However, leaning, and displaced vegetation provided further indication of slope movement.

General observations made at the south bank during the visual field inspection indicated the presence of over steepened slopes, progressive slope failure at localized areas along the bank indicative of deep slope instabilities, shallow slope instabilities near the bank toe, and erosion scarps at the toe of the bank. Identification of the slope instability mechanisms (i.e. tension cracks, bulging, scarps, etc.) could not be identified in detail due to the dense brush cover at the time of the inspection.

3.4.1 Bank Slope Observations

3.4.1.1 Northern Bank

- The ground to the north of the tree line and riverbank crest was a relatively flat field that is used as a Manitoba Hydro right-of-way.
- Within the western portion of the study area, the slope profile changes at the bank crest near the tree line, sloping sharply towards the river at approximately 1.5H:1V to 2H:1V before flattening out to 3H:1v to 4H:1V above the observed bank toe scarp. Within the eastern portion of the study area, the slope profiles changes at the bank crest near the tree line, and slopes towards the river at approximately 2H:1V to 2.5H:1V.
- The exposed bank slopes were generally covered by dense shrubs, bushes, and mature trees.
- Riprap was not observed within the study area.
- Within the western portion of the study area, scarps were observed near the bank crest in over steepened areas, indicative of shallow and/or deep-seated slope instabilities. These scarps typically ranged in vertical height from 300 mm to 900 mm.
- Within the eastern portion of the study area, scarps were observed at various locations along the bank, indicative of shallower slope instabilities. These scarps were typically 300 mm in vertical height.
- Erosion scarps were observed at the toe of the banks, ranging in vertical height from 300 mm to 600 mm
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.4.1.2 Southern Bank

- The ground to the south of the tree line and riverbank crest was a relatively flat field that is used as a Manitoba Hydro right-of-way.
- Within the western portion of the study area, the slope profile changes at the bank crest near the tree line, sloping sharply towards the river at approximately 2H:1V. Within the eastern portion of the study area, the slope profiles changes at the bank crest near the tree line, and slopes towards the river at approximately 2H:1V to 2.5H:1V.
- The exposed bank slopes were generally covered by dense shrubs, bushes, and mature trees.
- Riprap was not observed within the study area.
- Within the western portion of the study area, a series of slope instabilities and scarps up the slope were observed, indicative of progressive shallow and deep slope instabilities propagating up the bank. These scarps typically ranged in vertical height from 600 mm to 900 mm. Shallow slope instabilities were also observed near the toe of the bank.
- Within the eastern portion of the study area, scarps were observed at various locations along the bank, indicative of shallower slope instabilities. These scarps were typically 300 mm in vertical height.
- Erosion scarps were observed at the toe of the banks, ranging in vertical height from 300 mm to 600 mm.

- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.4.2 Existing Structures

3.4.2.1 Northern Bank

- No structures were observed within the study area.

3.4.2.2 Southern Bank

- No structures were observed within the study area.

3.4.3 Vegetation

3.4.3.1 Northern Bank

- Mowed lawn north of the tree line within the Manitoba right-of-way.
- The bank slopes were heavily vegetated with large mature trees and dense brush and shrub cover.
- Trees within the bank and along the bank crest were observed to be leaning towards the drain to varying degrees. The severity of the leaning was typically most noticeable in over steepened bank areas within the western portion of the study area.

3.4.3.2 Southern Bank

- Mowed lawn south of the tree line within the Manitoba right-of-way.
- The bank slopes within the western portion of the study area were heavily vegetated with large mature trees and dense brush and shrub cover, while the bank slopes within the eastern portion of the study were observed to be similar but with less mature trees.
- Trees within the bank slopes in close proximity observed slope instabilities were observed to be leaning towards the drain.

3.4.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-3: Summary of SCG and ECG Values (Site 6A)

Bank	SCG	ECG	Comments
North	2	2	Evidence of slope instabilities and erosion. However, asset installed deep within banks. Therefore, short-term potential for further deterioration of asset due to slope instability and erosion is low.
South	2	2	Evidence of slope instabilities and erosion. However, asset installed deep within banks. Therefore, short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.5 Site 6B: Dakota Feeder Main (Seine River)

General observations made at the west bank during the visual field inspection indicated minor erosion scarps at the riverbank toe and a very gradually sloping riverbank. There was no evidence of shallow or deep-seated failures along this bank. Based on the background information review and results of the visual field inspection the west bank would be appropriately classified as an erosion-controlled bank.

General observations made at the east bank during the visual field inspection indicated localized minor erosion scarps at the riverbank toe and a moderately sloped riverbank. There was no evidence of deep-seated failures along this bank. Based on the background information review and results of the visual field inspection the east bank would be appropriately classified as a failure-controlled bank.

3.5.1 Riverbank Slope Observations

3.5.1.1 Western Riverbank

- The ground surface slopes very gently eastward towards the Seine River.
- The riverbank profile has very little change in slope and was relatively flat up to approximately 2 m from the river edge, at which point the slope steepens to approximately 3H:1V to 4H:1V.
- The exposed bank slopes were generally covered by dense shrubs, bushes, and large mature trees.
- Riprap was not observed within the study area.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- Erosion scarps were observed at localized areas along the riverbank toe with a vertical height of approximately 300 mm.
- Animal burrows were frequently noted within the riverbank slope.

3.5.1.2 Eastern Riverbank

- The ground surface generally slopes westward towards the Seine River
- Within the southern portion of the study area, the slope profile is very gradual from the bank crest to approximately 5 m from the river edge, at which point the slope steepens to approximately 4H:1V to 5H:1V. The exposed riverbank slope was primarily covered in dense shrubs and bushes.
- Within the northern portion of the study area, the slope profile is relatively flat from the bank crest to approximately 10 m from the river edge, at which point the slope steepens to approximately 3H:1V down towards the river edge. The exposed bank slope was generally covered by dense shrubs, bushes, and large mature trees.
- Riprap was not observed within the study area.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- Erosion scarps were observed at localized areas along the riverbank toe with a vertical height of approximately 300 mm.
- Animal burrows were frequently noted within the riverbank slope.

3.5.2 Existing Structures

3.5.2.1 Western Riverbank

- No structures were observed within the study area.

3.5.2.2 Eastern Riverbank

- No structures were observed within the study area.

3.5.3 Vegetation

3.5.3.1 Western Riverbank

- The riverbank slopes were heavily vegetated with large mature trees, dense brush, and shrubs within the relatively flat portion of the riverbank slope. Closer to the edge of the river, brush and shrub remained dense while the presence of large mature trees became less frequent.

- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.5.3.2 Eastern Riverbank

- Within the southern portion of the study area, mowed lawn was observed east of the riverbank crest, with dense brush and shrubs being observed within the area between the riverbank crest and the river edge.
- Within the northern portion of the study area, the riverbank slopes were heavily vegetated with large mature trees, dense brush, and shrub.
- Some downed trees were observed in the vicinity of the crossing location but were broken part way up the trunk. It is unlikely that this occurred due to slope instability or erosion activities. Slight leaning of some trees towards the river was observed.
- There was no indication of significant vegetation movement resulting from slope instability within the study area.

3.5.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-4: Summary of SCG and ECG Values (Site 6B)

Bank	SCG	ECG	Comments
West	1	2	No defects observed with slope condition. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.
East	1	2	No defects observed with slope condition. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.6 Site 7: Rouge Road Feeder Main (Sturgeon Creek)

At the time of the visual field inspection, the level within Sturgeon Creek was much higher than typical conditions noted within the as-built documents. This was due to the presence of a beaver dam approximately 80 m south of the crossing location. As a result, much of the lower creek banks were not exposed at the time of the inspection, and observations were made based on the visible portions of the banks.

General observations made at the west bank during the visual field inspection indicated the presence of reasonably gradual slopes, becoming steeper close to the bridge abutment. There was no evidence of shallow or deep-seated failures along this bank, and minor erosion was observed at the creek edge. Grouted riprap was present around the bridge abutment side and head slopes as well as the exposed riverbank at the crossing location. Riprap was not observed within the study area to the south of the crossing location. Based on the background information review and results of the visual field inspection the west bank would be appropriately classified as an altered bank given the apparent slope regrading and riprap armouring likely completed during construction of the bridge structure and possibly the Sturgeon Creek Greenway Trail.

General observations made at the east bank during the visual field inspection indicated the presence of very gradual slopes becoming steeper close to the bridge abutment. There was no evidence of shallow or deep-seated failures along this bank, and minor erosion was observed at the creek edge. Grouted riprap was present around the bridge abutment side and head slopes as well as the exposed riverbank at the crossing location. Riprap was not observed within the study area to the south of the crossing location. Based on the background information review and results of the visual field inspection the west bank would be appropriately classified as an altered bank given the apparent slope regrading and riprap armouring likely completed during construction of the bridge structure.

3.6.1 Bank Slope Observations

3.6.1.1 Western Bank

- The ground surface south of the Hamilton Avenue bridge along the Sturgeon Creek Greenway Trail slopes gradually southeastward towards the creek. Part way down the bank slope the trail splits, with the northern leg sloping northeastward below the bridge and towards the creek, while the southern leg slopes southeastward towards the creek.
- The northern portion of the study area included much of the bridge infrastructure and west of the trail was observed to have steeper bridge abutment side slopes (approximately 3H:1V to 2H:1V with grouted riprap on the steeper portions) and a more gradual abutment head slope (approximately 2.5H:1V to 3H:1V) beneath the bridge to the west of the trail. To the east of the trail, the exposed bank was observed to be fairly flat.
- A crack was observed near the bank crest west of the bridge abutment. This area was observed to be frequented by bicycle traffic, and the crack is likely the result of desiccation of the near-surface soils rather than slope instability.
- The southern portion of the study area consisted of gently-sloping ground from the bank crest down towards the north-south oriented portion of the trail (approx. 6H:1V), becoming flatter at the trail, and then very gradually steepening down towards the creek edge.
- The crossing alignment is approximately at the interface between the northern and southern study areas described above.
- The upper portion of the exposed bank slope (west of the trail) was generally covered in mowed grass (and grouted rip rap in specific areas near the bridge), while the lower portion (east of the trail) is covered with brush.
- Within the northern portion of the study area, stone riprap was observed on the steeper bridge abutment side slopes, the entirety of the bridge head slope (west of the trail), and along the exposed portion of the bank slope east of the trail. Cracking of the grout (oriented in various directions) was observed at various locations within the grouted riprap areas.
- Riprap was not observed within the southern portion of the study area.
- Erosion scarps were not observed near the exposed bank toe within the northern portion of the study area.
- Erosion scarps and localized erosion gulley areas were observed along the exposed bank toe within the southern portion of the study area. These scarps ranged in vertical height from 100 mm to 450 mm.
- No evidence of deep-seated slope instabilities was noted within the bank slopes.
- A beaver dam was observed approximately 50 m south of the crossing location along the bank edge, and a beaver dam was located approximately 80 m south of the crossing location within the creek.

3.6.1.2 Eastern Bank

- The ground surface south of the Hamilton Avenue bridge sloped very gradually southwestward towards the creek. Slopes were observed to be steeper along the rear property lines of the houses further east, but these slopes are considered to be outside of the study area.
- The northern portion of the study area included much of the bridge infrastructure and west of the pedestrian trail that loops below the bridge was observed to have steeper bridge abutment side slopes (approximately 3H:1V to 2H:1V with grouted riprap on the steeper portions) and a more gradual abutment head slope (approximately 2.5H:1V to 3H:1V) beneath the bridge to the east of the trail. To the west of the trail, the exposed bank was observed to be fairly flat.

- The southern portion of the study area consisted of very gradual ground slope leading to the creek edge.
- The crossing alignment is approximately at the interface between the northern and southern study areas described above.
- The majority of the bank was covered in mowed grass (and grouted rip rap in specific areas near the bridge), while the lower portion consisted of brush.
- Within the northern portion of the study area, stone riprap was observed on the steeper bridge abutment side slopes, the entirety of the bridge head slope (west of the trail), and along the exposed portion of the bank slope west of the trail. Cracking of the grout oriented in various directions was observed at various locations within the grouted riprap areas.
- Riprap was not observed within the southern portion of the study area.
- Erosion scarps were not observed near the exposed bank toe within the northern portion of the study area.
- Erosion scarps and localized erosion gulley areas were observed along the exposed bank toe within the southern portion of the study area. These scarps ranged in vertical height from 100 mm to 450 mm.
- No evidence of deep-seated slope instabilities was noted within the bank slopes.
- A beaver dam was observed approximately 80 m south of the crossing location.

3.6.2 Existing Structures

3.6.2.1 Western Bank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structure - including superstructure and substructures (abutment and piers)
    - Manhole - MTS, located on sidewalk parallel to bridge
    - Light Post
    - Wood Post Barriers
    - Concrete Sidewalk – Parallel to Hamilton Avenue Bridge
    - Sidewalk – Sturgeon Creek Greenway Trail
    - Houses – Located southwest of crossing area and had chain link fenced-in backyard.
- Minor cracking of the concrete sidewalk pavement around the MTS manhole was observed (oriented in various directions).
- The trail pavement showed some signs of distress in localized areas within the study area. Cracks within the asphalt surface were generally orientated in a north south direction running approximately parallel to the creek.
- All other structures outlined above visually appeared in good condition.

3.6.2.2 Eastern Bank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structure - including superstructure and substructures (abutment and piers)
    - Manhole - MTS, located on sidewalk parallel to bridge
    - Concrete Sidewalk – Parallel to Hamilton Avenue Bridge
    - Sidewalk – Under-bridge walkway
- Minor cracking of the concrete sidewalk pavement around the MTS manhole was observed (oriented in various directions).
- The under-bridge sidewalk pavement showed minor signs of distress within the study area.
- All other structures outlined above visually appeared in good condition.

3.6.3 Vegetation

3.6.3.1 Western Bank

- Within the northern portion of the study area, the majority of the exposed slopes are covered with grouted riprap with minor vegetation growth occurring within the grout cracks.
- Within the southern portion of the study area, mowed lawn was observed west of the portion of the Sturgeon Creek Greenway trail that runs parallel to the creek. To the east of this trail, the vegetation consisted primarily of dense brush.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.6.3.2 Eastern Bank

- Within northern portion of the study area, majority of the exposed slopes are covered with grouted riprap with minor vegetation growth occurring within the grout cracks.
- Within the southern portion of the study area, mowed lawn was observed for the majority of the bank, becoming dense brush approximately 10 m east of the creek edge.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.6.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-5: Summary of SCG and ECG Values (Site 7)

Bank	SCG	ECG	Comments
West	2	2	Damming of the creek caused elevated creek levels and inability to see much of lower banks. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.
East	2	2	Damming of the creek caused elevated creek levels and inability to see much of lower banks. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.7 Site 8: West End Feeder Main (Omand's Creek)

General observations made at the west bank during the visual field inspection indicated the presence of fairly steep slopes directly against the bridge abutment that quickly transition into gradual slopes southward from the bridge. There was no evidence of shallow or deep-seated failures along this bank within the entire study area, and minor erosion was observed at the creek edge. Riprap was observed along an approximately 10 to 15 m length of the bank measured from the bridge abutment, with no riprap observed along the bank south of the abutment. Based on the background information review and results of the visual field inspection, the west bank would be appropriately classified as an altered bank given the slope regrading and riprap armouring that was completed during construction of the bridge structure.

General observations made at the east bank during the visual field inspection indicated the presence of fairly steep slopes directly against the bridge abutment that quickly transition into gradual slopes southward from the bridge near the crossing location, becoming steeper again further south of the crossing location. There was evidence of shallow slope instabilities in over steepened portions of un-armoured bank several meters south of the crossing location, and minor erosion was observed at the creek edge. Riprap was observed along an approximately 10 to 15 m length of the bank measured from the bridge abutment, with no riprap observed along the bank south of the abutment. Based on the background information review and results of the visual field inspection the east bank would be appropriately classified as an altered bank given the slope regrading and riprap armouring that was completed during construction of the bridge structure.

3.7.1 Bank Slope Observations

3.7.1.1 Western Bank

- The riprap amoured portion of the bank within the study area extended approximately 10 to 15 m from the bridge abutment, and was observed to have steeper slopes (approximately 2.5H:1V) near the bridge wingwall that quickly flattened out to 3.5H:1V to 4H:1V southward from the bridge. The riprap was generally large (greater than 600 mm).
- South of the riprap amoured portion of the bank within the study area, the slopes were observed to be approximately 3H:1V to 4H:1V. The bank crest is located adjacent to a paved roadway and is nearly flat.
- The crossing alignment is within the riprap amoured area of the bank.
- Riprap is located along the entirety of the exposed bank face (from crest to toe). In non-amoured areas, the bank slope was covered with dense brush. A portion of the bank crest was vegetated with packed-down grass (area between bank crest and Empress Street), while the remainder of the bank crest is a relatively flat, paved street (Empress Street).
- A narrow crack was observed along the bank crest within the grassed area between the bank crest and Empress Street This area was observed to be frequented by bicycle traffic, and the crack was more likely the result of desiccated surface soils and not a sign of slope instability.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- Erosion scarps were not observed near the exposed bank toe within the riprap amoured area. Minor erosion was observed within the non-amoured portion of the exposed bank toe, although the dense brush cover in this area made detailed visual inspection difficult.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.7.1.2 Eastern Bank

- The riprap armoured portion of the bank within the study area extended approximately 10 to 15 m from the bridge abutment, and was observed to have steeper slopes (approximately 2.5H:1V) near the bridge wingwall that quickly flattened out to 3.5H:1V to 4H:1V southward from the bridge. The riprap was generally large (greater than 600 mm).
- South of the riprap armoured portion of the bank within the study area, the slopes were observed to be over steepened at various locations, ranging from 2H:1V to 3H:1V. The bank crest was generally flat and extended into a private property driveway/parking lot immediately east of the site.
- The crossing alignment is within the riprap armoured area of the bank.
- Where observed, the riprap was located along the entirety of the exposed bank face (from crest to toe). In non-armoured areas, the bank slope was covered with dense brush. Brush and clusters of large mature trees were observed between the bank crest and the fence line of the neighboring property for the entirety of the study area.
- Localized slope instabilities were observed at various locations within the study area south of the riprap armoured banks. A scarp ridge was observed near the bank crest immediately south of the riprap with a vertical height of 75 mm, and underlying organic soils were exposed at ground surface in this area (brush vegetation was scarce).
- Erosion scarps were not observed near the exposed bank toe within the riprap armoured area. Minor erosion was observed within the non-armoured portion of the exposed bank toe, although the dense brush cover in this area made detailed visual inspection difficult.

- Animal burrows were frequently observed within the bank slope and crest south of the riprap armoured area.

3.7.2 Existing Structures

3.7.2.1 Western Bank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structure - including superstructure and substructures (abutment, wingwall)
    - Hydro pole
    - Paved street – Empress Street
    - Street Signage – Stop Sign
- All structures outlined above visually appeared in good condition.

3.7.2.2 Eastern Bank

- The following structures were observed within and adjacent to the study area:
    - Bridge Structure - including superstructure and substructures (abutment)
    - Hydro pole
    - Granular Parking Lot – Private property east of creek
    - Chain Link Fence – Along edge of private property east of creek
- Hydro pole was approximately vertical, although an angled wood post support was observed to be leaning against the south side of the hydro pole to provide additional support. However, given that the wood post was supporting the hydro pole on the south side (support parallel to the bank crest), it is unlikely that past leaning of the hydro pole was related to the slope stability of the bank.
- All other structures outlined above visually appeared in good condition.

3.7.3 Vegetation

3.7.3.1 Western Bank

- Within the armoured portion of the study area, minor vegetation was observed through riprap along bank slope. A partially grassed area was observed between curb of Empress Street and bank crest.
- Outside of the armoured portion of the study area, dense brush vegetation was observed along the bank slope. A partially grassed area was observed between curb of Empress Street and bank crest.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.7.3.2 Eastern Bank

- Within the armoured portion of the study area, some vegetation growth was observed through riprap along the bank slope. The bank crest was comprised of dense brush and clusters of mature trees.
- Outside of the armoured portion of the study area, dense brush vegetation was observed along the bank slope. The bank crest was comprised of dense brush and clusters of large mature trees.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.7.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-6: Summary of SCG and ECG Values (Site 8)

Bank	SCG	ECG	Comments
West	1	2	No defects observed with slope condition. Minor erosion observed south of riprap armoured slope within study area. Short-term potential for further deterioration of asset due to slope instability and erosion is low.
East	2	2	Evidence of slope instabilities and minor erosion observed south of riprap armoured slope within study area. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.8 Site 9: West End Feeder Main (Truro Creek)

General observations made at the west bank during the visual field inspection indicated the presence of gradual to very gradual slopes from the bank crest (Assiniboine Golf Course) down to the creek. There was no evidence of shallow or deep-seated failures along this bank within the entire study area, and minor erosion was observed at the creek edge. Based on the background information review and results of the visual field inspection the west bank would be appropriately classified as an altered bank given the slope regrading that appears to have been done during construction of the feeder main, and likely during development of the Assiniboine Golf Course.

General observations made at the east bank during the visual field inspection indicated the presence of gradual to very gradual slopes from the bank crest (Silver Avenue) down to the creek. There was no evidence of shallow or deep-seated failures along this bank within the entire study area, and minor erosion was observed at the creek edge. Based on the background information review and results of the visual field inspection the west bank would be appropriately classified as an altered bank given the slope regrading that appeared to have been done during construction of the feeder main, and likely during development around Silver Avenue.

3.8.1 Bank Slope Observations

3.8.1.1 Western Bank

- The ground surface within the Assiniboine Golf Course is approximately flat, with a gentle southeastward slope towards Truro Creek.
- The bank profile within the study area changes from approximately flat along the crest (within the Assiniboine Golf Course) to a slope of approximately 4H:1V from the bank crest down to the creek edge.
- The exposed bank slopes around the crossing alignment were generally covered by shrubs, bushes, and some maturing trees.
- North of the crossing alignment, a pedestrian bridge (Silver Avenue Pathway) crosses Truro Creek. The banks of Truro Creek within 10 m of this bridge structure were observed to be graded at approximately 4H:1V and have a geotextile separator fabric as well as riprap armouring along the entirety of the slope face. The riprap was medium sized (less than 300 mm).
- Approximately half of the riprap along this bank was observed to be displaced down the slope, leaving a large area of exposed geotextile close to the bridge abutment. This may be due to an insufficient coefficient of friction between the fabric and the slope soil material.
- Riprap was not observed south of the riprap armoured banks near the bridge structure.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- Erosion scarps were not observed near the exposed bank toe within the riprap armoured area at the bridge. Minor erosion was observed within the non-armoured portion of the exposed bank toe, although the dense brush cover in this area made detailed visual inspection difficult.
- Animal burrows were frequently noted within the riverbank slope.

3.8.1.2 Eastern Bank

- The ground surface west and north of Silver Avenue within the study area has a gentle northwestern slope towards Truro Creek.
- The bank profile within the study area changes from a very gradual slope along the crest (area north of Silver Avenue) to a slope of approximately 4H:1V from the bank crest down to the creek edge.
- The bank crest primarily consisted of mowed grass, while the exposed bank slope was generally covered by shrubs, bushes, and some maturing trees down to the creek edge.
- North of the crossing alignment, a pedestrian bridge (Silver Avenue Pathway) crosses Truro Creek. The banks of Truro Creek within 10 m of this bridge structure were observed to be graded at approximately 4H:1V and have a geotextile separator fabric as well as riprap armouring along the entirety of the slope face. The riprap was medium sized (less than 300 mm).
- A small fraction of the riprap along this bank was observed to be displaced down the slope.
- Riprap was not observed south of the riprap armoured banks near the bridge structure.
- No evidence of shallow or deep-seated slope instabilities were noted within the bank slope.
- Erosion scarps were not observed near the exposed bank toe within the riprap armoured area at the bridge. Minor erosion was observed within the non-armoured portion of the exposed bank toe, although the dense brush cover in this area made detailed visual inspection difficult.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.8.2 Existing Structures

3.8.2.1 Western Bank

- The following structures were observed within and adjacent to the study area:
  - Pedestrian Bridge Structure - including superstructure and substructures (abutments)
  - Fence – Heavily damaged
  - Geotechnical Instrument – Pneumatic Piezometer (RST Instruments)
- The fence was observed to be heavily damaged down the bank. It is highly unlikely that this damage was incurred as a result of slope instabilities.
- All other structures outlined above visually appeared in good condition.

3.8.2.2 Eastern Bank

- The following structures were observed within and adjacent to the study area:
  - Pedestrian Bridge Structure - including superstructure and substructures (abutments)
  - Paved Roadway – Silver Avenue
  - Paved Pedestrian Walkway – Silver Avenue Pathway
  - Traffic Signage
- All structures outlined above visually appeared in good condition.

3.8.3 Vegetation

3.8.3.1 Western Bank

- Mowed grass was observed beyond the bank crest within limits of the Assiniboine Golf Course. The upper bank slopes were moderately vegetated with brush, shrubs, and maturing trees. Closer to the edge of the creek, the density of brush and shrub increased while the presence of maturing trees became less frequent.
- The riprap armoured banks in close proximity to the bridge did not show signs of vegetation growth through the geotextile fabric or riprap.

- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.8.3.2 Eastern Bank

- Mowed grass was observed along the bank crest (north and west of Silver Avenue) right up to the point where the bank slopes start to steepen. The bank slopes were densely vegetated with brush, shrubs, and some clusters of maturing trees.
- The riprap armoured banks in close proximity to the bridge did not show signs of vegetation growth through the geotextile fabric or riprap.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.8.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-7: Summary of SCG and ECG Values (Site 9)

Bank	SCG	ECG	Comments
West	1	2	No defects observed with slope condition. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.
East	1	2	No defects observed with slope condition. Minor erosion observed. Short-term potential for further deterioration of asset due to slope instability and erosion is low.

3.9 Site 10: Haney-Moray Feeder Main (Assiniboine River)

General observations made at the north bank during the visual field inspection indicated the presence of scarps of varying height partway up the riverbank, likely due to a combination of riverbank erosion and shallow-seated slope instabilities driven by the erosion. There was no evidence of deep-seated or rotational failures along this bank. Riprap was not observed along the banks, although cobbles and boulders were observed within the study area near the bank toe. The gradually sloping nature of the area suggests that regrading work was likely done during construction of the William R. Clement Parkway bridges and associated pedestrian pathways. Therefore, the north bank would be appropriately classified as an altered bank.

General observations made at the south bank during the visual field inspection indicated the presence of scarps of varying height near the river edge, likely due to a combination of riverbank erosion and shallow seated slope instabilities driven by the erosion. Slope instabilities were also observed within over steepened portions of the riverbank within the eastern portion of the study area and at a localized area in close proximity to the crossing alignment. Riprap was observed in localized areas along the bank toe in close proximity to the crossing location, and cobbles and boulders were also observed within the study area near the bank toe. The gradually sloping nature of the area and the presence of a tree clearing along the feeder main alignment suggests that regrading work was likely done during construction of the feeder main and William R. Clement Parkway bridges. Therefore, the south bank would be appropriately classified as an altered bank.

3.9.1 Riverbank Slope Observations

3.9.1.1 Northern Riverbank

- The riverbank crest within the study area reaches a peak height in an area near the pedestrian staircase located at the north abutment of the east William R. Clement Parkway bridge. From this point, the slope gradually starts to increase to a slope of approximately 3.5H:1V until reaching an east-west oriented pedestrian pathway where the bank slope flattens out. To the south of the pedestrian pathway, the slope steepens to approximately 3H:1V down to an observed scarp approximately 2 to 3 m from the river edge. The exposed bank slope between the base of the observed scarp and the river edge was approximately 3H:1V.
- Between the observed scarp and the river edge vegetation was primarily absent, and exposed glacial soils were observed.
- Stone riprap was not observed along the banks, although cobbles and boulders were observed within the study area along the bank toe.
- Scarps were noted approximately 2 to 3 m away from the river edge, indicative of potential erosion and/or shallow slope instabilities. These scarps typically ranged in vertical height from 300 mm to 900 mm within the study area (smaller to the west, larger to the east).
- No evidence of deep-seated slope instabilities was noted within the riverbank slope.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.9.1.2 Southern Riverbank

- A gently sloping clearing through forested areas was observed along the crossing alignment leading northward towards the riverbank crest.
- Within the western portion of the study area, the riverbank crest sloped gently down towards the river, steepening slightly approximately 10 m south of an observed scarp near the river edge, and flattening out again approximately 2 m south of the scarp. The exposed bank slope between the base of the observed scarp and the river edge was approximately 3H:1V to 4H:1V.
- Within the eastern portion of the study area, the riverbank crest sloped very gently down towards the river, reaching a ground surface elevation approximately 1 to 2 m higher than that of the western portion of the study area. At a distance of approximately 4 m from the observed scarp at the river edge, the bank slope steepens to approximately 2H:1V, flattening out again approximately 0 to 1 m south of the scarp. The exposed bank slope between the base of the observed scarp and the river edge was approximately 3H:1V to 4H:1V.
- Between the observed scarp and the river edge vegetation was primarily absent, and exposed glacial soils were observed.
- Within the western portion of the study area large scarps were noted approximately 2 m away from the river edge, indicative of potential erosion and/or shallow slope instabilities. These scarps typically ranged in vertical height from 600 mm to 900 mm. A small scarp and tension crack were also observed approximately 2 m south of the large scarp within the flattened portion of the riverbank, indicative of potential slope instability. This smaller scarp had a vertical height of approximately 75 mm.
- Within the eastern portion of the study area a large scarp was noted approximately 2 m way from the river edge, indicative of potential erosion and/or shallow slope instabilities. This scarp typically ranged in vertical height from 600 mm to 900 m. An additional scarp was observed approximately 1 m south of the large scarp where the over steepened bank flattened out. This scarp had a vertical height of approximately 200 mm. Another larger scarp was observed slightly further east approximately 3 m south of the large scarp, and had a vertical height of approximately 600 mm. The instabilities noted in this area appeared to be indicative of progressive slope instability moving southward up the over steepened portion of the riverbank.

- Stone riprap was observed at localized locations near the bank toe in close proximity to the crossing location. Cobbles and boulders were observed within the study area along the bank toe.
- No evidence of animal burrows or infestations were noted within the riverbank slope.

3.9.2 Existing Structures

3.9.2.1 Northern Riverbank

- The following structures were observed within and adjacent to the study area:
  - Bridge Structures (2) - including superstructure and substructures (abutments and piers)
  - Drainage Culverts– CSP Outfall
  - Light Posts
  - Pavement Sidewalk
  - Steel Safety Barriers along Sidewalk Edge
  - Masonry Retaining Walls
  - Chain Link Fence – Along private property east of study area
  - Information Sign
- Some blocks within the masonry retaining walls were observed to have undergone small movements. In general, the walls are in good condition.
- All other structures outlined above visually appeared in good condition.

3.9.2.2 Southern Riverbank

- The following structures were observed within and adjacent to the study area:
  - Bridge Structures (2) - including superstructure and substructures (abutments and piers)
  - Chain Link Fence – Along private property east of study area (oriented north-south)
  - Farm Fence – Along private property east of study area (oriented east-west)
  - House – Located east of study area
- The farm fence was located within the eastern portion of the study area within the area undergoing progressive slope instabilities due to oversteepening. The farm fence supports were generally observed to be leaning towards the river.
- All other structures outlined above visually appeared in good condition.

3.9.3 Vegetation

3.9.3.1 Northern Riverbank

- The upper portion of the riverbank slope (north of the pedestrian pathway) was generally covered in mowed grass with some clusters of large mature trees. The lower portion of the riverbank slope (south of the pedestrian pathway) was generally covered in moderately dense brush, shrubs, and local clusters of large trees. Further east of the study area, the density of large trees increased.
- There was no indication of significant vegetation movement that would suggest slope instability within the study area.

3.9.3.2 Southern Riverbank

- The western portion of the study area was characterized by mowed grass along the bank crest within the cleared crossing alignment, and dense brush, shrubs, and clusters of mature trees along the bank west of the cleared area. Vegetation was primarily absent in the exposed bank area to the north of the observed scarp near the river edge.

- The eastern portion of the study area was characterized by dense brush, shrubs, and large trees. Vegetation was primarily absent in the exposed bank area to the north of the observed scarp near the river edge.
- Within the eastern portion of the study area, trees within the over steepened bank slope were observed to be leaning towards the river to varying degrees. Trees located north of the observed slope instabilities (founded within the failed soil masses) generally leaned more severely towards the river than those south of the observed instabilities.
- Within the western portion of the study, the vegetation did not show any indication of significant movement resulting from slope instability.

3.9.4 SCG and ECG Values

The following table provides a brief summary of the SCG and ECG ratings selected for each bank at this site. Additional information regarding selection of these values is provided within **Appendix D**.

Table 3-8: Summary of SCG and ECG Values (Site 10)

Bank	SCG	ECG	Comments
North	2	2*	Evidence of erosion. Absence of available geotechnical information indicated need for investigation and further analysis. Geotechnical investigation at this site completed and results presented in Section 4. Slope stability analysis completed at this site and results presented in Section 5.
South	2*	2*	Evidence of slope instabilities and erosion. Absence of available geotechnical information indicated need for investigation and further analysis. Geotechnical investigation at this site completed and results presented in Section 4. Slope stability analysis completed at this site and results presented in Section 5.

Notes: *Selected ratings revised from “3” to “2” following completion of the geotechnical investigation and slope stability analyses discussed in subsequent sections

4. Geotechnical Investigation

4.1 General

Based on the results of the background information review and the visual field inspection, the following two sites were determined to require geotechnical investigation, laboratory testing, and instrumentation installation/monitoring:

- Site 5: West Perimeter Force Main (Assiniboine River)
- Site 10: Haney-Moray Feeder Main (Assiniboine River)

For Site 5, the intent of the geotechnical investigation was to provide subsurface information and soil testing to support other disciplines in completion of their pipeline inspection as part of the project scope. For Site 10, the intent of the geotechnical investigation was to provide subsurface information and soil testing to be used in preliminary slope stability analyses to determine the minimum factor of safety of a slip surface intersecting the pipeline, as the north bank was characterized as having an ECG of 3 and the south bank was characterized as having an SCG and ECG of 3.

A job hazard assessment was prepared prior to the geotechnical investigation, and public utility clearance certificates at both sites were obtained by AECOM personnel from representatives of ClickBeforeYouDigMB and DigShaw. Subsurface conditions observed during drilling were documented by AECOM geotechnical personnel, and recovered samples were classified according to the Modified Unified Classification System for soils. Other pertinent information such as groundwater and drilling conditions were also recorded during the field investigation.

4.2 Site 5: West Perimeter Force Main (Assiniboine River)

On January 25, 2021 two (2) test holes (TH21-01 and TH21-02) were drilled at the approximate locations shown on **Figure E1** in **Appendix E**. Drilling was completed by Maple Leaf Drilling Ltd. using a Mobile B54X drill rig equipped with 125 mm Solid Stem Augers (SSA's) to a maximum depth of 6.4 m below ground surface (BGS). Standard penetration tests (SPT) were performed at select depths within both test holes. Disturbed grab and split spoon samples and relatively undisturbed Shelby Tube samples were retrieved from test holes at select intervals. Upon completion of the drilling, standpipe piezometers were installed in both test holes.

Samples retrieved during the field investigation were tested in AECOM's Materials Testing Laboratory (soil index tests) and ALS Environmental's Materials Testing Laboratory (soil electrochemical tests), both located in Winnipeg, Manitoba.

Detailed test hole logs have been prepared for each test hole and are attached as **Appendix F**. The test hole logs include descriptions and depths of the soil units encountered, sample type, sample location, results of field and laboratory testing and other pertinent information such as seepage and sloughing related to groundwater conditions.

Table 4-1 summarizes the location, elevation, and depth of each test hole.

Table 4-1: Test Hole Information Summary (Site 5)

Test Hole ID	Northing (m)	Easting (m)	Surface Elevation (m)	Termination Depth (m BGS)
TH21-01	5525507	620346	233.85	6.40
TH21-02	5525365	620348	231.90	5.33

4.2.1 Laboratory Testing

Laboratory soil testing was conducted on select soil samples collected during the geotechnical investigation. The soil testing program included the determination of moisture content, grain size distribution (hydrometer/sieve analysis), Atterberg Limits, bulk unit weight, and undrained shear strength ("QU/2" unconfined compressive strength, "PP" pocket penetrometer, and "TV" Torvane methods). The electrochemical testing program included determination of resistivity/conductivity, sulphate content, pH, and chloride content. The laboratory test results are presented in **Appendix G**.

Table 4-2 summarizes the number of each test completed, and **Figure 4-1** illustrates the variation in moisture content and Atterberg Limits with depth.

Table 4-2: Summary of Laboratory Testing (Site 5)

Test	Number
SPT's	5
Moisture Content	15
Atterberg Limits	5
Grain Size Distribution (Hydrometer/Sieve Analysis)	4
Undrained Shear Strength (QU/2)	1
Undrained Shear Strength (PP)	2
Undrained Shear Strength (TV)	2
Bulk Unit Weight	1
Electrochemical (Resistivity/Conductivity, Sulphate, pH, Chloride)	6

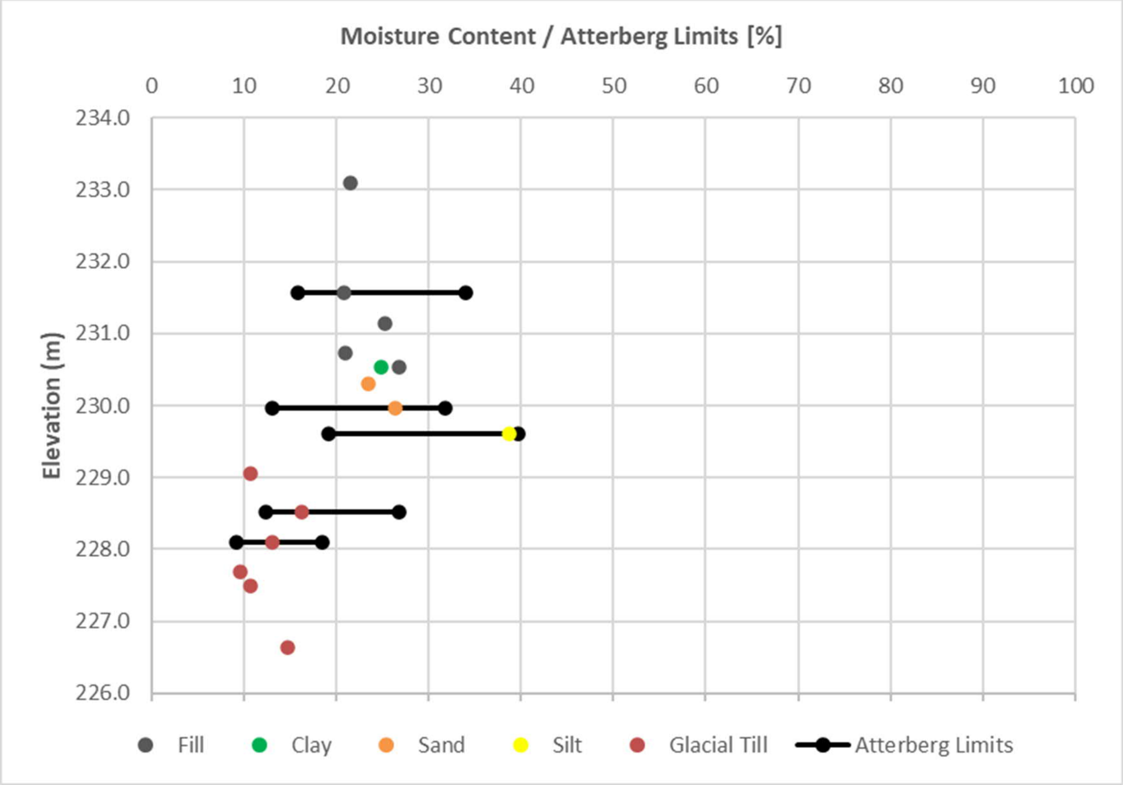


Figure 4-1 - Summary of Moisture Content and Atterberg Limits vs. Depth (Site 5)

4.2.2 Subsurface Conditions

The following sections describe the subsurface conditions encountered during the geotechnical investigation at Site 5. Information provided in this section is a summary of the findings from the investigation and laboratory testing.

In descending order from grade, the general soil profile consisted of:

- Topsoil (Fill)
- Fill
- Clay
- Sand
- Silt
- Glacial Till

Each of these units are described separately below.

Topsoil (Fill)

A layer of topsoil was encountered at ground surface in both test holes and was approximately 0.1 m thick. The topsoil was black and frozen at the time of the investigation. It was placed as part of finish grading during prior construction.

Fill

A layer of fill was encountered beneath the topsoil in both test holes, and ranged in thickness from 1.4 m to 3.2 m. In test hole TH21-01 the fill layer was classified as clay at depths ranging from 0.1 m to 0.9 m, sand from 0.9 m to 1.1 m, and silt from 1.1 m to 3.2 m. In test hole TH21-02 the fill layer was classified as clay from 0.1 m to 1.5 m.

The clay fill was generally silty, contained some sand, trace gravel, trace roots, was brown to grey, and was classified as firm to stiff, moist, and of intermediate to high plasticity at depths below 0.9 m. At depths above 0.9 m, the clay fill was frozen at the time of the investigation. Suspected cobbles were encountered during drilling of test hole TH21-02 at a depth of 1.2 m. A summary of the index properties of the clay fill is presented in **Table 4-3**.

Table 4-3: Summary of Index Properties of Clay Fill (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	22	27	3
Undrained Shear Strength, PP (kPa)	60		1
Undrained Shear Strength, TV (kPa)	39		1

The sand fill was silty, contained trace to some clay, and was brown and frozen at the time of the investigation.

The silt fill was sandy, clayey, brown to mottled dark brown, firm, moist, and of intermediate plasticity. A summary of the index properties of the silt fill is presented in **Table 4-4**.

Table 4-4: Summary of Index Properties of Silt Fill (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	21		2
SPT ‘N’ Blow Count (uncorrected)	5		1
Atterberg – Plastic Limit (%)	16		1
Atterberg – Liquid Limit (%)	34		1
Grain Size – Gravel (%)	0		1
Grain Size – Sand (%)	24		1
Grain Size – Silt (%)	53		1
Grain Size – Clay (%)	23		1

Clay

A layer of native clay was encountered beneath the fill in test hole TH21-01 with an approximate thickness of 0.3 m. The clay was silty, contained trace to some sand, and was brown, soft to firm, moist, and of intermediate plasticity. A summary of the index properties of the clay is presented in **Table 4-5**.

Table 4-5: Summary of Index Properties of Clay (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	25		1
Undrained Shear Strength, QU/2 (kPa)	22		1
Undrained Shear Strength, PP (kPa)	36		1
Undrained Shear Strength, TV (kPa)	34		1
Bulk Unit Weight (kN/m^3)	19.1		1

Sand

A layer of sand was encountered beneath the clay in test hole TH21-01 with an approximate thickness of 1.0 m. The sand was silty, clayey, brown to grey, firm, moist to wet, and of intermediate plasticity. A summary of the index properties of the sand is presented in **Table 4-6**.

Table 4-6: Summary of Index Properties of Sand (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	24	26	2
Atterberg – Plastic Limit (%)	13		1
Atterberg – Liquid Limit (%)	32		1
Grain Size – Gravel (%)	0		1
Grain Size – Sand (%)	44		1
Grain Size – Silt (%)	30		1
Grain Size – Clay (%)	26		1

Silt

A layer of silt was encountered beneath the fill in test hole TH21-02 with an approximate thickness of 1.2 m. The silt was clayey, contained some sand, and was brown to mottled grey, soft to firm, moist, and of intermediate plasticity. A summary of the index properties of the silt is presented in **Table 4-7**.

Table 4-7: Summary of Index Properties of Silt (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	39		1
Atterberg – Plastic Limit (%)	19		1
Atterberg – Liquid Limit (%)	40		1
Grain Size – Gravel (%)	0		1
Grain Size – Sand (%)	13		1
Grain Size – Silt (%)	58		1
Grain Size – Clay (%)	30		1

Glacial Till

A layer of glacial till was encountered beneath the sand in test hole TH21-01 and beneath the silt in test hole TH21-02 at depths of 4.4 m and 2.7 m below ground surface, respectively. Both test holes were terminated within the glacial till layer due to auger refusal at depths ranging from 5.3 m to 6.4 m. The glacial till was generally classified as silty sand containing some gravel, some clay, and was light brown, firm to hard, dry to wet, and of low plasticity. A summary of the index properties of the glacial till is presented in **Table 4-8**.

Table 4-8: Summary of Index Properties of Glacial Till (Site 5)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	10	16	6
SPT 'N' Blow Count (uncorrected)	6	>50	4
Atterberg – Plastic Limit (%)	9	12	2
Atterberg – Liquid Limit (%)	19	27	2
Grain Size – Gravel (%)	19		1
Grain Size – Sand (%)	46		1
Grain Size – Silt (%)	20		1
Grain Size – Clay (%)	15		1

4.2.3 Sloughing and Groundwater Conditions

Sloughing was not encountered within test holes TH21-01 or TH21-02 during drilling. Seepage was not encountered in test hole TH21-02 but was observed during drilling of TH21-01 at depths below 4.6 m. Detailed information about the nature and location of the sloughing and/or seepage are provided on the test hole logs included in **Appendix F**.

Two (2) standpipe piezometers were installed in test holes TH21-01 and TH21-02. Short-term monitoring results of the groundwater level (GWL) are provided in **Table 4-9**.

Table 4-9: Piezometer Monitoring Data (Site 5)

Test Hole Number	TH21-01	TH21-02
Test Hole Elevation [m]	233.85	231.90
Tip Depth [m BGS]	6.25	2.44
Tip Elevation [m]	227.60	229.46
Tip Location	Glacial Till	Silt
Dates	**GWL Depth Below Ground Surface (Elevation) [m]**	
***January 25, 2021**	5.85 (228.00)	2.15 (229.75)
February 22, 2021	4.22 (229.62)	2.18 (229.72)

* Measurements taken immediately following installation

It should be noted that groundwater levels, seepage, and sloughing levels in excavations may vary seasonally, annually, or as a result of construction activities.

4.2.4 Electrochemical Test Results

Electrochemical testing was completed on six (6) soil samples collected from test holes TH21-01 and TH21-02 to determine water soluble sulphate in soil, pH of soil, water soluble chloride in soil, and soil resistivity/conductivity. A summary of the test results is provided in **Table 4-10**.

Table 4-10 – Summary of Electrochemical Tests (Site 5)

Soil Unit	Borehole	Sample ID / Depth (m)	Water Soluble Sulphate (mg/kg)	pH	Water Soluble Chloride (mg/kg)	Resistivity (ohm*cm)	Conductivity (mS/cm)
Clay Fill	TH21-01	G1 / 0.8	35	7.49	373	1210	0.824
	TH21-02	G1 / 0.8	58	7.65	64	1940	0.515
Sand	TH21-01	G5 / 3.8	118	7.76	306	1330	0.750
Silt	TH21-02	G3 / 2.3	128	7.67	116	1710	0.584
Glacial Till	TH21-01	S8 / 6.2	76	8.10	132	2420	0.414
	TH21-02	S6 / 4.4	177	8.03	120	1700	0.587

The results of the water-soluble sulphate testing indicate that the clay fill, sand, and silt soils tested are classified as moderate (S-3) class of exposure to sulphate attack according to CAN/CSA A23.1-M94 (*Concrete Materials and Methods of Concrete Construction*). However, it is known that alluvial and glaciolacustrine soils in the Winnipeg area commonly have a very severe (S-1) class of exposure to sulphate attack.

Based on the results of the resistivity/conductivity testing, the clay fill, sand, and silt soils tested are classified as highly corrosive to buried metal.

4.3 Site 10: Haney-Moray Feeder Main (Assiniboine River)

On January 26, 2021 two (2) test holes (TH21-03 and TH21-04) were drilled at the approximate locations shown on **Figure E2** in **Appendix E**. Drilling was completed by Maple Leaf Drilling Ltd. using a Mobile B54X drill rig equipped with 125 mm Solid Stem Augers (SSA's) to a maximum depth of 5.3 m below ground surface (BGS). Standard penetration tests (SPT) were performed at select depths within both test holes. Disturbed grab and split spoon samples and relatively undisturbed Shelby Tube samples were retrieved from the test holes at select intervals. Upon completion of the drilling, standpipe piezometers were installed in both test holes.

Samples retrieved during the field investigation were tested in AECOM's Materials Testing Laboratory (soil index tests) and ALS Environmental's Materials Testing Laboratory (soil electrochemical tests), both located in Winnipeg, Manitoba.

Detailed test hole logs have been prepared for each test hole and are attached as **Appendix F**. The test hole logs include descriptions and depths of the soil units encountered, sample type, sample location, results of field and laboratory testing and other pertinent information such as seepage and sloughing related to groundwater conditions.

Table 4-11 summarizes the location, elevation, and depth of each test hole.

Table 4-11: Test Hole Information Summary (Site 10)

Test Hole ID	Northing (m)	Easting (m)	Surface Elevation (m)	Termination Depth (m BGS)
TH21-03	5525903	624809	231.90	5.33
TH21-04	5525799	624792	229.78	3.35

4.3.1 Laboratory Testing

Laboratory soil testing was conducted on select soil samples collected during the geotechnical investigation. The soil testing program included the determination of moisture content, grain size distribution (hydrometer/sieve analysis), Atterberg Limits, bulk unit weight, and undrained shear strength (“QU/2” unconfined compressive strength, “PP” pocket penetrometer, and “TV” Torvane methods). The electrochemical testing program included determination of resistivity/conductivity, sulphate content, pH, and chloride content. The laboratory test results are presented in **Appendix G**.

Table 4-12 summarizes the number of each test completed, and **Figure 4-2** illustrates the variation in moisture content and Atterberg Limits with depth.

Table 4-12: Summary of Laboratory Testing (Site 10)

Test	Number
SPT’s	4
Moisture Content	12
Atterberg Limits	4
Grain Size Distribution (Hydrometer/Sieve Analysis)	4
Undrained Shear Strength (QU/2)	1
Bulk Unit Weight	1
Electrochemical (Resistivity/Conductivity, Sulphate, pH, Chloride)	5

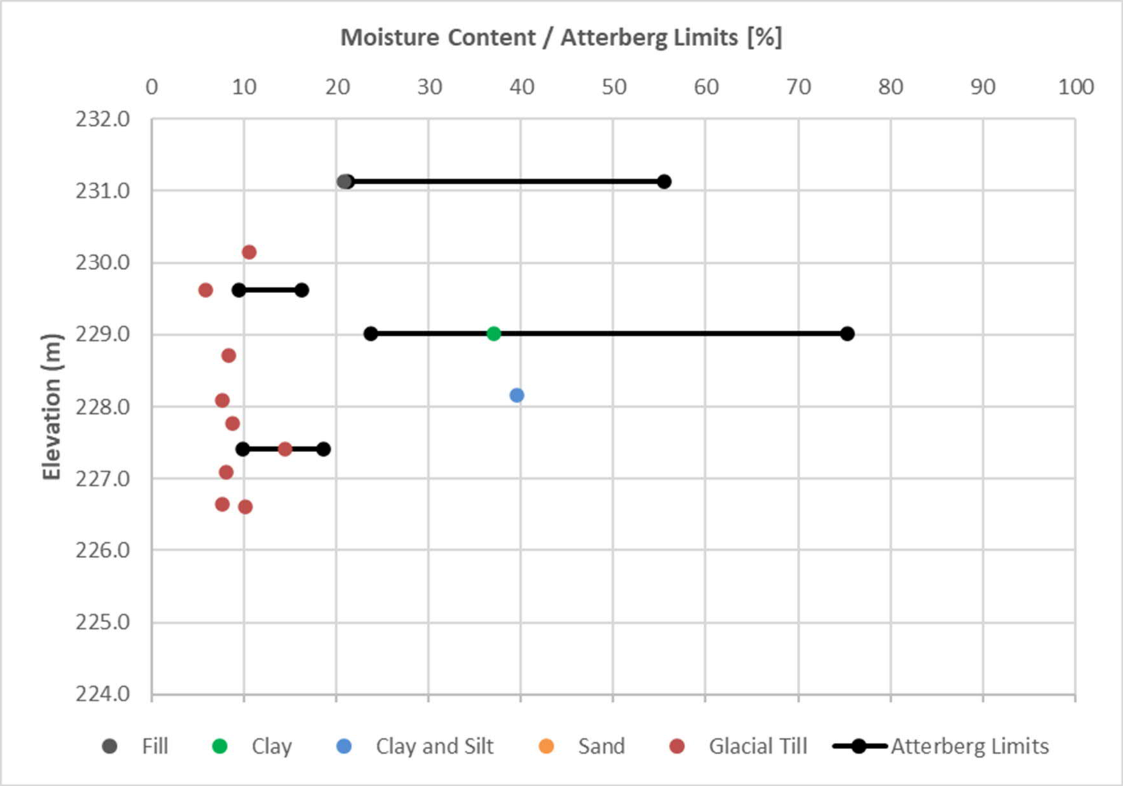


Figure 4-2 - Summary of Moisture Content and Atterberg Limits vs. Depth (Site 10)

4.3.2 Subsurface Conditions

The following sections describe the subsurface conditions encountered during the geotechnical investigation at Site 10. Information provided in this section is a summary of the findings from the investigation and laboratory testing.

In descending order below grade, the general soil profile consisted of:

- Topsoil (Fill)
- Clay and Silt (Fill)
- Clay
- Clay and Silt
- Sand
- Glacial Till

Each of these units are described separately below.

Topsoil (Fill)

A layer of topsoil was encountered at ground surface in both test holes and was approximately 0.1 m thick. The topsoil was black and frozen at the time of the investigation. It was placed as part of finish grading during prior construction.

Clay and Silt Fill

A layer of clay and silt fill was encountered beneath the topsoil in test hole TH21-03 with a thickness of 0.9 m. The clay and silt fill generally contained some sand, trace gravel, trace roots, and was dark brown and frozen at the time of the investigation. A summary of the index properties of the clay and silt fill is presented in **Table 4-13**.

Table 4-13: Summary of Index Properties of Clay and Silt Fill (Site 10)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	21		1
Atterberg – Plastic Limit (%)	21		1
Atterberg – Liquid Limit (%)	56		1
Grain Size – Gravel (%)	1		1
Grain Size – Sand (%)	18		1
Grain Size – Silt (%)	30		1
Grain Size – Clay (%)	51		1

Clay

A layer of native clay was encountered beneath the topsoil in test hole TH21-04 with an approximate thickness of 1.1 m. The clay was silty, contained trace roots, and was brown, frozen to 1.1 m, and firm, moist, and of high plasticity below 1.1 m. A summary of the index properties of the clay is presented in **Table 4-14**.

Table 4-14: Summary of Index Properties of Clay (Site 10)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	37		1
Atterberg – Plastic Limit (%)	24		1
Atterberg – Liquid Limit (%)	75		1
Grain Size – Gravel (%)	0		1
Grain Size – Sand (%)	0		1
Grain Size – Silt (%)	21		1
Grain Size – Clay (%)	79		1

Clay and Silt

A layer of clay and silt was encountered beneath the clay in test hole TH21-04 with an approximate thickness of 0.5 m. The clay and silt were grey, firm, moist, and of high plasticity. A summary of the index properties of the clay and silt is presented in **Table 4-15**.

Table 4-15: Summary of Index Properties of Clay and Silt (Site 10)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	40		1

Sand

A layer of sand was encountered beneath the clay and silt in test hole TH21-04 with an approximate thickness of 0.2 m. The sand contained some clay to clayey, trace silt, and was grey to mottled brown, firm, moist, and of low plasticity.

Glacial Till

A layer of glacial till was encountered beneath the clay fill in test hole TH21-03 and beneath the sand in test hole TH21-04 at depths of 0.9 m and 1.9 m below ground surface, respectively. Both test holes were terminated within the glacial till layer due to auger refusal at depths ranging from 3.4 m to 5.3 m. The glacial till was generally classified as sand and silt containing some clay, trace to some gravel, and was light brown, soft to hard, dry to moist, and of low plasticity. Suspected cobbles or boulders were encountered during drilling of test hole TH21-04 at a depth of 2.4 m. A summary of the index properties of the glacial till is presented in **Table 4-16**.

Table 4-16: Summary of Index Properties of Glacial Till (Site 10)

Test	Minimum Value	Maximum Value	Number of Tests
Moisture Content (%)	6	14	9
SPT 'N' Blow Count (uncorrected)	46	>50	4
Atterberg – Plastic Limit (%)	9	10	2
Atterberg – Liquid Limit (%)	16	19	2
Grain Size – Gravel (%)	6	16	2
Grain Size – Sand (%)	37	39	2
Grain Size – Silt (%)	35	38	2
Grain Size – Clay (%)	12	18	2
Undrained Shear Strength, QU/2 (kPa)	24		1
Bulk Unit Weight (kN/m^3)	23.5		1

4.3.3 Sloughing and Groundwater Conditions

Sloughing and seepage were not encountered within test holes TH21-03 or TH21-04 during drilling. Detailed information about the nature and location of the sloughing and/or seepage are provided on the test hole logs included in **Appendix F**. Two (2) standpipe piezometers were installed in test holes TH21-03 and TH21-04. Short-term monitoring results of the groundwater level (GWL) are provided in **Table 4-17**.

Table 4-17: Piezometer Monitoring Data (Site 10)

Test Hole Number	TH21-03	TH21-04
Test Hole Elevation [m]	231.90	229.78
Tip Depth [m BGS]	5.18	3.05
Tip Elevation [m]	226.72	226.73
Tip Location	Glacial Till	Glacial Till
Dates	**GWL Depth Below Ground Surface (Elevation) [m]**	
***January 26, 2021**	Dry (-)	Dry (-)
February 22, 2021	Dry (-)	1.99 (227.79)

* Measurements taken immediately following installation

It should be noted that groundwater levels, seepage, and sloughing depth in excavations may vary seasonally, annually, or as a result of construction activities.

4.3.4 Electrochemical Test Results

Electrochemical testing was completed on five (5) soil samples collected from test holes TH21-03 and TH21-04 to determine water soluble sulphate in soil, pH of soil, water soluble chloride in soil, and soil resistivity/conductivity. A summary of the test results is provided in **Table 4-18**.

Table 4-18 - Summary of Electrochemical Tests (Site 10)

Soil Unit	Borehole	Sample ID / Depth (m)	Water Soluble Sulphate (mg/kg)	pH	Water Soluble Chloride (mg/kg)	Resistivity (ohm*cm)	Conductivity (mS/cm)
Clay and Silt Fill	TH21-03	G1 / 0.8	21	7.44	32	2400	0.416
Clay	TH21-04	G1 / 0.8	126	7.83	<20	2040	0.489
Glacial Till	TH21-03	S4 / 3.2	192	8.14	35	2860	0.350
	TH21-03	G7 / 5.3	112	8.10	21	3190	0.313
	TH21-04	S4 / 3.2	62	8.03	27	3790	0.264

The results of the water-soluble sulphate testing indicate that the clay and silt fill, clay, and glacial till soils tested are classified as moderate (S-3) class of exposure to sulphate attack according to CAN/CSA A23.1-M94 (*Concrete Materials and Methods of Concrete Construction*). However, it is known that alluvial and glaciolacustrine clay soils in the Winnipeg area commonly have a very severe (S-1) class of exposure to sulphate attack.

With respect to buried metal, based on the results of the resistivity/conductivity testing, the clay and silt fill and clay encountered at this site are highly corrosive, and the glacial till encountered is corrosive to highly corrosive.

5. Slope Stability Assessment

5.1 General

The primary objective of the preliminary slope stability analysis is to assess the existing stability of the river/creek bank slopes determined to have an SCG and/or ECG value greater than or equal to 3, and to determine if prevailing slope conditions place the buried sewer/water systems at increased risk of damage from slope movement. Based on the results of the background information review and visual field inspection, slope stability analyses have been completed for the following two sites:

- Site 4: Fort Garry/St Vital Interceptor Siphons (Red River) – West Riverbank
- Site 10: Haney-Moray Feeder Main (Assiniboine River) – North and South Riverbanks

5.2 Limitations of Slope Stability Analyses

The primary objective of the stability assessment was to establish the levels of risk to the buried pipes at the crossings as a result of slope instability within the banks and is not necessarily a characterization of the stability of the banks themselves. Furthermore, slope stability analysis has been performed for each site based upon in some cases limited or old topographical information (i.e., LIDAR data and as-built record information), and limited pipe invert/condition information and positional information. The results should therefore be viewed as preliminary.

5.3 Methodology

5.3.1 Stability Analysis

Two-dimensional slope stability models were developed using GeoStudio 2019 (Slope/W) based on the Limit Equilibrium method of analysis. The riverbank geometries were established based on LIDAR survey provided by the City (City of Winnipeg 2011 Data Set), as-built record drawings, and existing geotechnical reports.

The soil stratigraphy for the stability models was derived from geological maps, available test hole information from previously existing geotechnical engineering reports, and information obtained from the geotechnical investigation completed as part of this project (for Site 10). The pipe location at each crossing was taken from the record drawings, and the pipe profiles within the slope stability models were inferred where necessary.

Upon establishing a slope stability model for each site, the assessment was performed using Morgenstern-Price's general method of slices, which satisfies both moment and horizontal force equilibrium. More advanced methods (such as finite element analysis) were not used for this study as the uncertainties associated with material parameters, soil stratigraphy and piezometric conditions would not justify a more complex analysis method.

As part of the analysis, the following slip surfaces were considered of interest and are conceptually illustrated in **Figure 5-1**. A Factor of Safety (FS) was determined for each of the following:

- **Global Slip Surface Engaging Pipe (GS+P)**: is defined as a slip surface that meets the criteria of a global slip surface and encompasses part of the buried pipe.
- **Global Slip Surface (GS)**: is defined as a slip surface that largely encompasses the slope soil mass and has an entry and exit point at or just beyond the slope crest and/or toe.
- **Toe Slip Surface (TS)**: is defined as a slip surface that is localized to the toe of the slope and which has a minimum depth of 0.5m. At some locations the FS of this slip surface may be lower than the critical or global FS. Instability at the toe of the slope may reduce the FS for the global or critical slip surfaces. Retrogressive failures starting at the toe will generally work towards the riverbank.

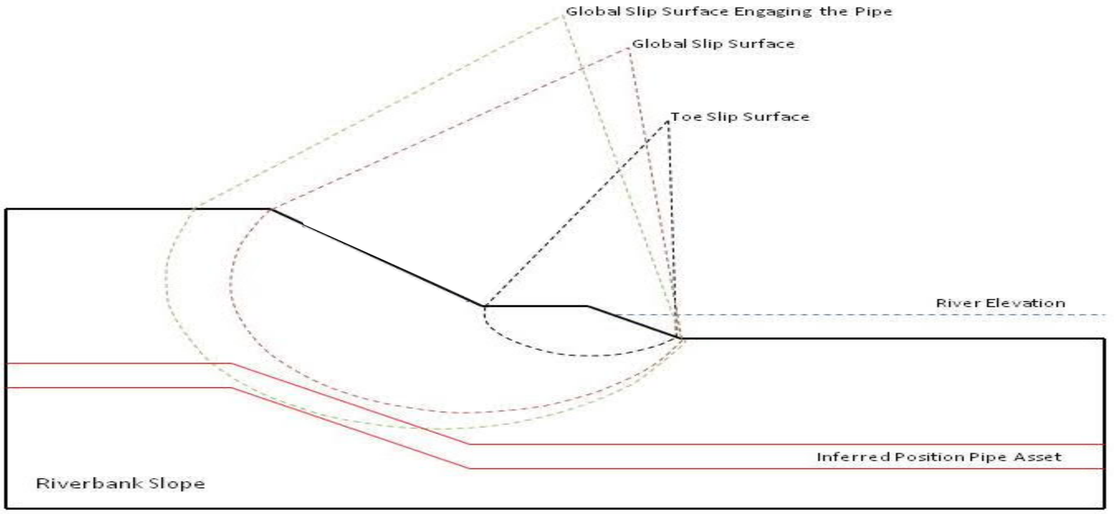


Figure 5-1 - Assessed Slip Surfaces

5.3.2 Slope Stability Cases

The following loading conditions have been considered as part of the slope stability analysis, and are outlined below:

- Long-term Conditions (Summer Water Level and Winter Water Level)
- Short-term Condition (Rapid Drawdown)

An acceptable FS can be defined between 1.3 and 1.5 depending on whether short-term or long-term conditions are being considered, and based on other factors including but not limited to associated impact of instability, risk management approach and related cost to improve the stability. For purposes of this TM and consistent with acceptable design practice, river/creek stability is assessed under the following design conditions and the corresponding target FS against slope instability:

- Long-term Condition: FS ≥ 1.50
- Short-term Condition (Rapid Drawdown): FS ≥ 1.30

The short-term rapid drawdown condition refers to a state in which the river level against the bank falls rapidly below its normal level while the piezometric conditions within the bank slope remain at their elevated levels.

5.3.3 Soil Parameters

Soil strength parameters used in the stability analyses are presented in **Table 5-1** and **Table 5-2** for Site 4 and Site 10, respectively. Soil parameters were selected based upon review of existing and collected laboratory testing data for each site, combined with local knowledge and prior experience.

5.3.3.1 Site 4: Fort Garry/St. Vital Interceptor Siphons (Red River)

In order to develop the slope stability model at the west riverbank, subsurface stratigraphy and groundwater conditions from the following available test hole logs were relied upon:

- **Test Holes 1003**, **1004, and 401**: Klohn Leonoff Consultants Ltd (April 12, 1976), *Report on Sub-Soils Investigation for Fort Garry- St. Vital Corridor, Winnipeg, Manitoba*. These test hole logs are included in **Appendix B1**.

Further information regarding the subsurface ground conditions at this site are shown on the as-built drawings attached in **Appendix A1**.

Fully-softened shear strength values were assigned to the alluvial and glaciolacustrine clay soil layers for both the long-term and short-term cases. The bedrock was treated as an impenetrable layer within the analyses, and therefore was not assigned a shear strength value. Riprap armouring at the toe of the west bank was not considered within the analyses, as available as-built records did not indicate the extent (lateral and vertical) of the armouring, and observations from the visual field inspection suggested that it was only present within a small area immediately around the crossing alignment. The following table summarizes the parameters adopted as part of the slope stability analysis.

Table 5-1: Soil Strength Parameters for Stability Analysis (Site 4)

Stratum	Bulk Unit Weight (kN/m^3)	Effective Angle of Internal Friction (Degrees)	Effective Cohesion (kPa)
Alluvial Clay*	18	18	5.0
Glaciolacustrine Clay	18	14	5.0
Glacial Till	21	30	10.0

Notes: *Inclusive of Upper and Lower Alluvial Clay.

5.3.3.2 Site 10: Haney-Moray Feeder Main (Assiniboine River)

In order to develop the slope stability model at the north and south riverbanks, subsurface stratigraphy and groundwater conditions were based on the geotechnical investigation completed by AECOM as part of this project.

Fully-softened shear strength values were assigned to the alluvial and glaciolacustrine soil layers for both the long term and short-term cases. The thickness of glacial till and bedrock contact depth were not confirmed during the drilling at this site. As such, it has been assumed that the glacial till layer extends from the contact elevation observed to the lowest elevation considered within the analysis. The following table summarizes the parameters adopted as part of the slope stability analysis at the site.

Table 5-2: Soil Strength Parameters for Stability Analysis (Site 10)

Stratum	Bulk Unit Weight (kN/m^3)	Effective Angle of Internal Friction (Degrees)	Effective Cohesion (kPa)
Clay and Silt Fill	18.5	18	2.0
Clay / Clay and Silt	18	14	5.0
Sand	21	32	0.0
Glacial Till	21	36	0.0

5.3.4 River Water Levels

Levels for the Red River modeled in the slope stability analysis for Site 4 were selected based on information from the City of Winnipeg's online database (http://www.winnipeg.ca/publicworks/pwddata/riverlevels/) as well previous geotechnical reports associated with the site. Levels for the Assiniboine River modeled in the slope stability analysis for Site 10 were selected based on river elevation information presented in the as-built record. The normal winter water level (NWWL), normal summer water level (NSWL), and rapid drawdown (RDD) heights incorporated into the slope stability analyses are summarized in **Table 5-3** below.

Table 5-3: Summary of River Levels for Stability Analysis

Water Course	Site Reference	NWWL (m)	NSWL (m)	*RDD (m)	Reference Document
Red River	Site 4	221.76	223.74	1.98	• City of Winnipeg Online Database Reference Levels Table
Assiniboine River	Site 10	227.84	228.40	0.56	• City of Winnipeg As-Built Drawing D-846

*Notes: Difference between NWWL and NSWL levels.

5.4 Slope Stability Results

5.4.1 Site 4: Fort Garry / St. Vital Interceptor Siphons (Red River)

Slope stability analyses were completed for the west bank of Site 4 based on the established subsurface ground model and available topographic information along the pipe alignment. The FS values calculated from the analyses are presented in **Table 5-4**.

Table 5-4: Current Riverbank Stability Results Along Pipe Alignment (Site 4)

Slope Stability Case	Global Slip Stability (GS)	Global Stability Engaging the Pipe (GS+P)	Toe Slip Surface (TS)	File Output Reference
	West	West	West	West
Long Term (NWWL)	1.39	1.39	1.39	H-01
Long Term (NSWL)	1.46	1.46	1.46	H-02
Short Term (RDD)	1.30	1.30	1.30	H-03

Based on the results of the preliminary slope stability assessment for Site 4, the following general conclusions and recommendations were drawn:

- For long-term analysis conditions (NWWL and NSWL) at the west bank, the 700 mm and 800 mm HDPE interceptor sewers are at risk of being engaged by a failure surface with a FS between 1.39 and 1.46. For short-term analysis conditions (RDD), the 700 mm and 800 mm HDPE interceptor sewers are engaged by a failure surface with a FS of 1.30.
- The short-term FS values meet the current industry accepted design standard FS of 1.30.
- Whilst the existing long-term FS values are somewhat below current industry-accepted design standards, the risk of immediate slope failure is considered low. A progressive reduction in the FS of the riverbank slope through erosion should be monitored regularly to mitigate the risk of reduction in slope stability through erosion.
- Consideration of slope improvements within the western riverbank should be assessed on a cost/benefit basis. Unless deemed critical, periodic visual inspection should be sufficient in the short term until such time that existing slope stability falls below a FS of about 1.3. Should the need for slope improvement to be required in the short term, consideration may be given to slope regrading and placement of stone riprap within a greater area around the crossing location.

5.4.2 Site 10: Haney-Moray Feeder Main (Assiniboine River)

Slope stability analyses were completed both banks of Site 10 based on the established subsurface ground model and available topographic information along the pipe alignment. The FS values calculated from the analyses for Site 10 are presented in **Table 5-5**.

Table 5-5: Current Riverbank Stability Results Along Pipe Alignment (Site 10)

River Conditions	Global Slip Stability (GS)		Global Stability Engaging the Pipe (GS+P)		Toe Slip Surface (TS)		File Output Reference	
	North	South	North	South	North	South	North	South
Long Term (NWWL)	2.60	1.83	2.60	>2.50	2.60	1.83	H-04	H-05
Long Term (NSWL)	2.60	1.84	2.60	>2.50	2.60	1.84	H-06	H-07
Short Term (RDD)	2.56	1.83	2.56	>2.50	2.56	1.83	H-08	H-09

Based on the results of the preliminary slope stability assessment for Site 10, the following general conclusions and recommendations were drawn:

- For long-term analysis conditions (NWWL and NSWL) and short-term analysis conditions (RDD) at both banks, the 450 mm CPP feeder main was engaged by failure surfaces with a FS greater than 2.50.
- The long-term and short-term FS values meet the current industry accepted design standard FS's of 1.50 and 1.30, respectively.
- Geotechnical investigation completed by AECOM as part of this project indicated that the pipe was installed at least partially within the glacial till unit. Therefore, slope instabilities observed along the south bank are shallow in nature and unlikely to damage the pipeline.
- Based on the slope stability results, the SCG and ECG values at the north bank (at this time) are more appropriately selected as 1 and 2, respectively.
- Based on the slope stability results, the SCG and ECG values at the south bank (at this time) are more appropriately selected as 2 and 2, respectively.
- No further action is required unless the slope conditions deteriorate or significantly different hydraulic conditions (river level) are experienced.

6. Closing

The findings and conclusions contained within this TM were based on the results of as-built records, information contained within previous studies, and for Sites 5 and 10, new subsurface investigations. In some cases, soil conditions and groundwater levels were extrapolated based on existing data and AECOM's prior experience. If conditions are encountered that appear to be different from those shown within the existing documentation and described in this report, or if assumptions stated herein are not in keeping with the design, this office should be notified in order that the recommendations can be review and justified, if necessary.

Soil conditions by their nature can be highly variable across a site. If conditions at any of the HRRC sites reviewed in this TM are encountered that appear to be different from those identified, or if the assumptions stated herein are not in keeping with the design and operations of the HRRC Crossings, this office should be notified in order to review and adjust (if necessary) the material contained within report.

If you have any questions, please do not hesitate to contact the undersigned.

Respectfully submitted,
AECOM Canada Ltd.

Prepared by:

PROVINCE OF MANITOBA
R.M. HARRAS
Member
34508
REGISTERED PROFESSIONAL ENGINEER

Ryan Harras

2021-03-17

Ryan Harras, B.Sc. (Civil), P.Eng
Geotechnical Engineer

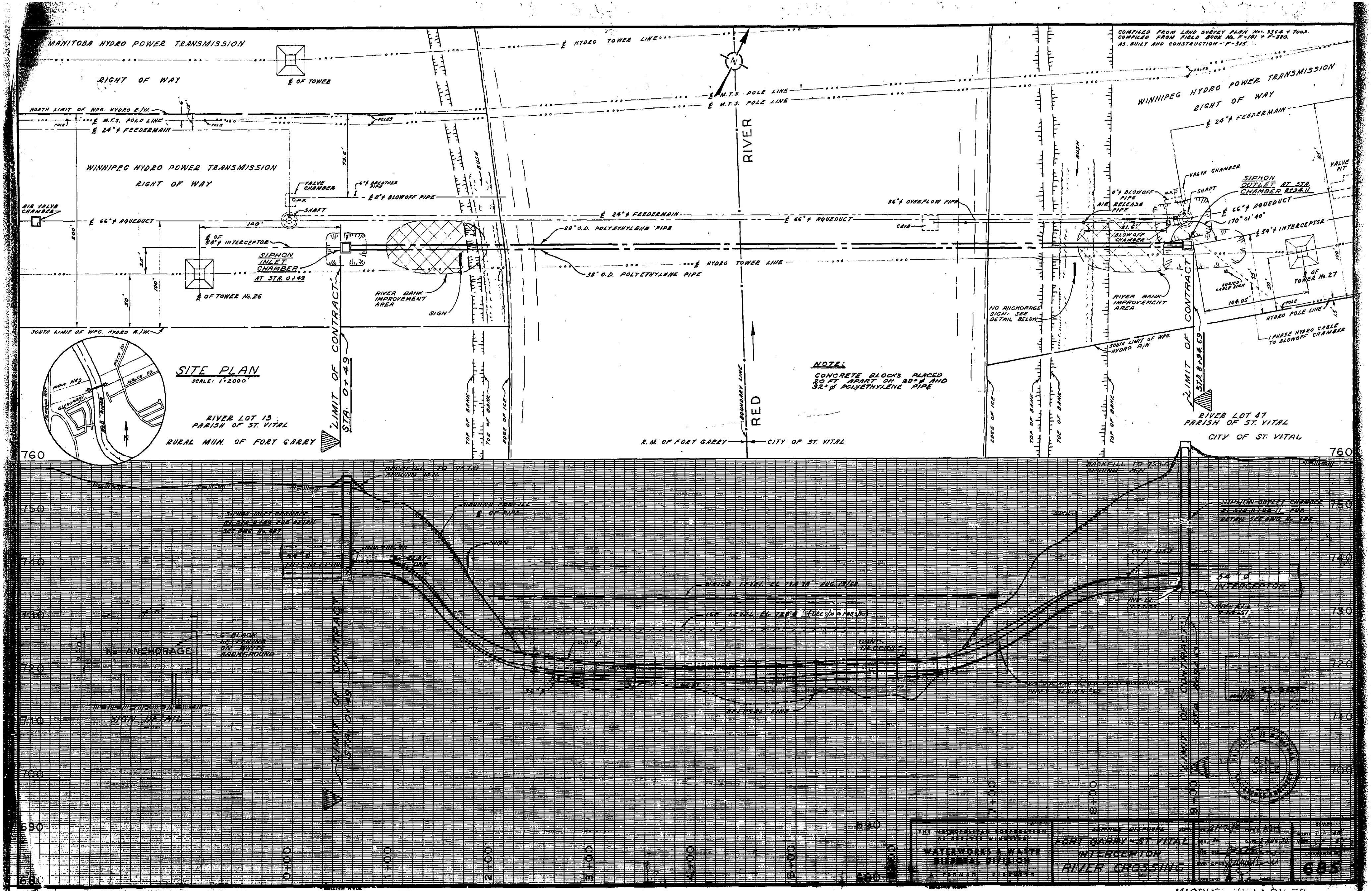
Reviewed by:

Elliott E. Drumright

Elliott Drumright, PhD, P.E
Associate Geotechnical Engineer

Appendix A

A1: Site 4 As-Built Records
A2: Site 5 As-Built Records
A3: Site 6 As-Built Records
A4: Site 7 As-Built Records
A5: Site 8 As-Built Records
A6: Site 9 As-Built Records
A7: Site 10 As-Built Records



THE METROPOLITAN CORPORATION OF GREATER WINNIPEG
WATERWORKS & WASTE DISPOSAL DIVISION

FORT GARRY - ST VITAL
INTERCEPTOR
RIVER CROSSING

685

MICROFILMED NOV 76

D SIZE 22" x 34" (559mm x 864mm)

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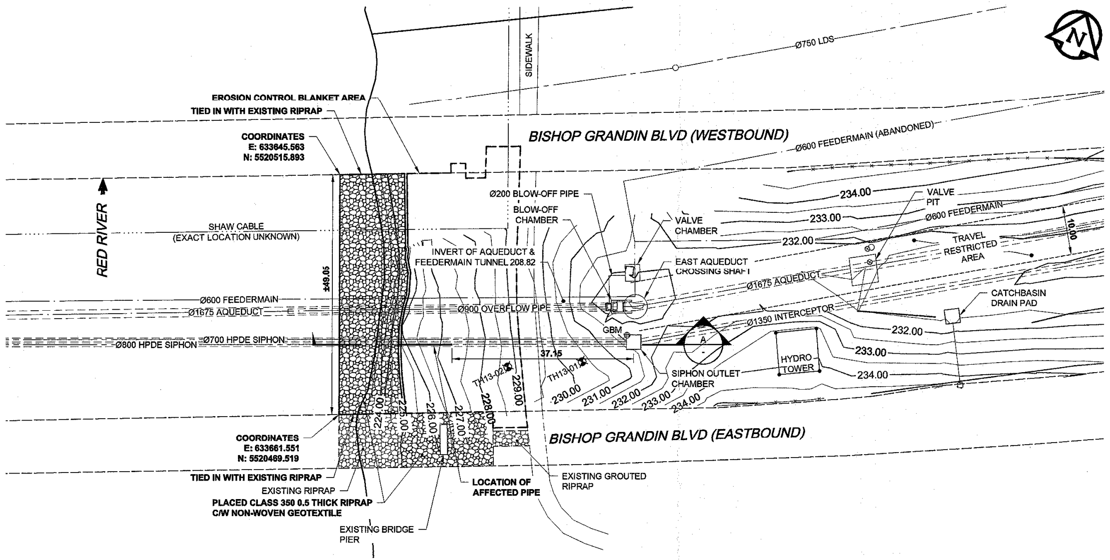

PLAN

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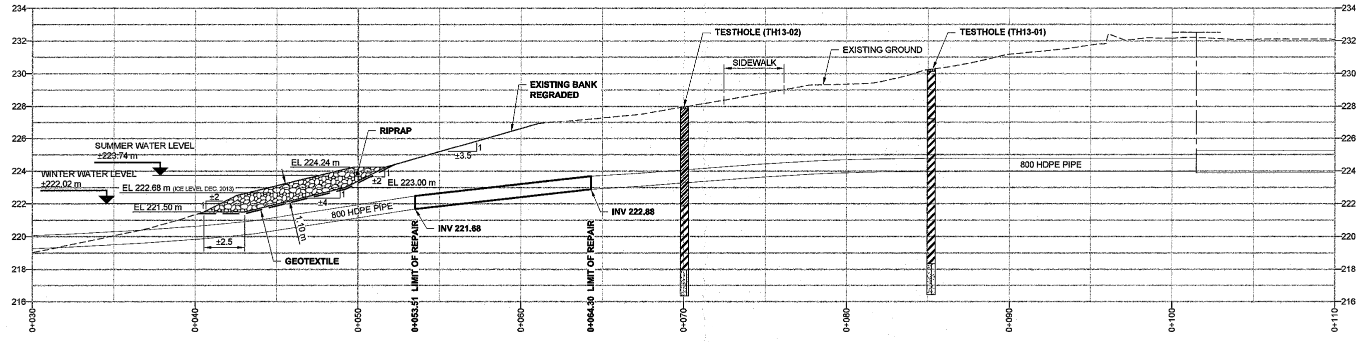


A SECTION

1:150

LEGEND

- - - - - -	EXISTING GROUND PROFILE	▨	EXISTING GROUND TO BE EXCAVATED
— — —	GEOTEXTILE	▦	RIP RAP CLASS 350
———	NEW GROUND PROFILE		
▩	TOPSOIL / ORGANICS	▨	CLAY
▨	CLAY AND SILT	▦	TILL

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

APEGM
Certificate of Authorization
AECOM Canada Ltd.
No. 4671 Date: June 14/16

RECORD DRAWING

BID OPPORTUNITY NO. 1040-2013

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PROFILE	NEW
150 WM	WATERMAIN	150 WM	♂	CURB STOP	●	150 WM	WATERMAIN	150 WM
○	HYDRANT	●	▷	REDUCER	▶	+	HYDRANT	+
⊗	VALVE	⊗	✕	COUPLING	✕	X	VALVE	X
300 LDS	LAND DRAINAGE SEWER	300 LDS	⤳	ANODE	⤳	300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS	— — —	HYDRO		250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●	— · —	MTS		— · —	PAVEMENT CROWN	— · —
□	CATCH BASIN	■	— ··· —	GAS		—◇—	N/W PROPERTY LINE	
△	CURB INLET	▲	⊕	TESTHOLE		—○—	S/E PROPERTY LINE	
⊏⊐	CULVERT	⊏⊐	•—•	LAMP STANDARD		—□—	N/W GUTTER	
—Ǝ	PIPE ABANDONMENTS	—Ǝ	⊙	TREE		— ⊖ —	S/E GUTTER	
⌖	SURVEY BAR							

AECOM REVIEW DRFT CHK

AECOM FILE NAME: 60274906-01-B-1002-R0X.dwg Saved By: coopera

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPR. U/G STRUCTURES COMMITTEE   DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE. BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

NO.	REVISIONS	YY/MM/DD	BY
1	ISSUED FOR RECORD	16/05/12	KMB/KLC
0	ISSUED FOR CONSTRUCTION	13/12/13	CM

AECOM

DESIGNED BY	AJH	CHECKED BY	
DRAWN BY	CM	APPROVED BY	
HOR. SCALE	AS NOTED	RELEASED FOR CONSTRUCTION	
VERT. SCALE	AS NOTED		
DATE		DATE	

PROFESSIONAL'S SEAL
Z. T. AL-HAYAZAI
Member 37236
June 14/16

CONSULTANT DRAWING NO.
60274906-01-B-1002-R0X.dwg

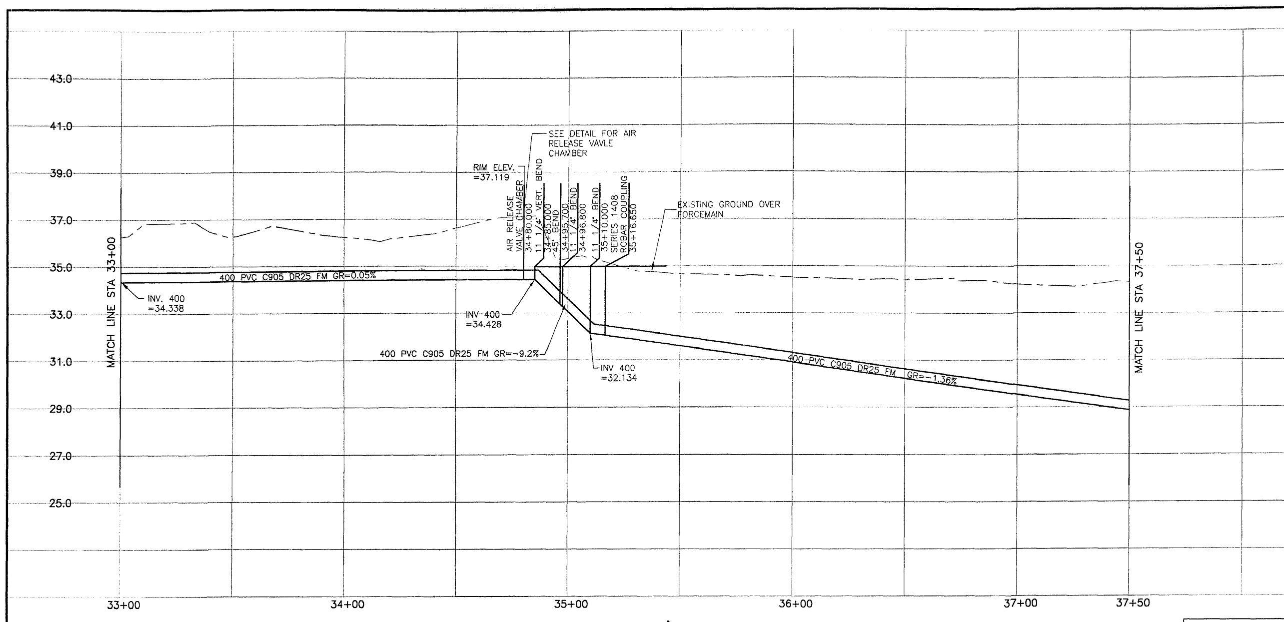
Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

FORT GARRY INTERCEPTOR
SIPHON REPAIR
GEOTECHNICAL
PLAN/PROFILE

SHEET 2 OF 2

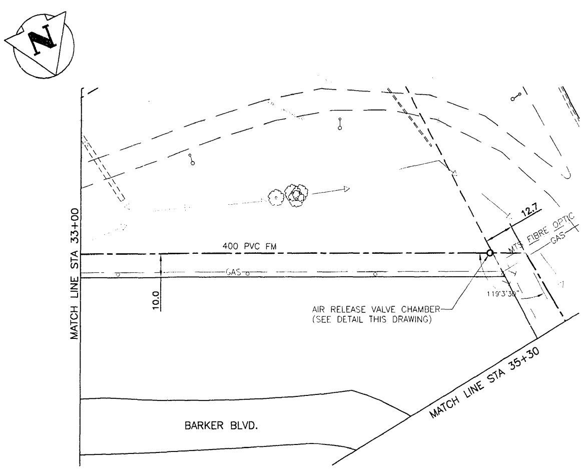
CITY DRAWING NUMBER
LD-7277
REV 1

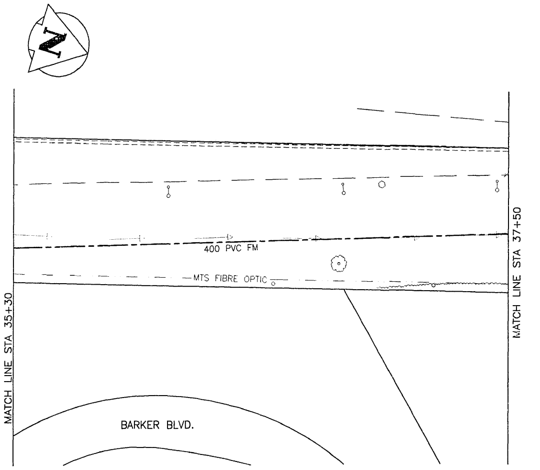


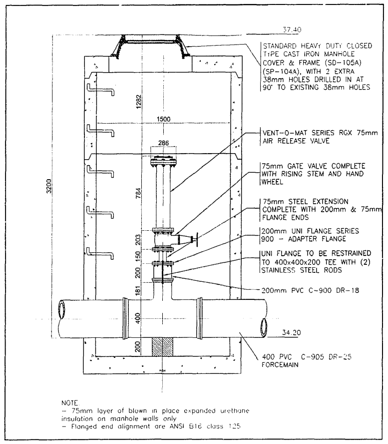
MATERIAL LIST

WASTE WATER SEWER	MANUFACTURER	SPECIFICATION	SPECIAL FEATURES
MANHOLE BASES	LAFARGE	ASTM C76	REINF. CONC.
MANHOLE RISERS	LAFARGE	ASTM C76	REINF. CONC.
FRAMES AND COVERS	TITAN FOUNDRY	CW 2131/CW 3205	C.I.
600 SEWER	IPEX	ASTM F769	PVC
600 OUTFALL SEWER	LAFARGE	ASTM C76	REINF. CONC.
400 FORCEMAIN PIPE	IPEX CENTURION	ASTM C-900	PVC
550 ENCASEMENT PIPE	CCTF (OAKVILLE, ONT.)	8.74mm THICK	STEEL
400 ENCASEMENT PIPE (RIVER CROSSING)	CCTF	9.53mm THICK	STEEL (DEV-TAR COATING)
600 CSP	ARMCO	2mm THICK	GALVANIZED METAL
600 SLIP JOINT	ARMCO		GALVANIZED METAL
400-45° BEND	GRINNELL	AWWA C-110	C.I.
400-22.5° BEND	IPEX	ASTM-C900	PVC
400 GATE VALVE	CLOW	AWWA C-509	
400-300 REDUCER	STAR	C-110	PVC
400-11.25° BEND	STAR	C-110	PVC
COUPLING	FORD		
400 COUPLING	ROBAR	C-110 DI	STEEL

CONTRACTOR: MAPLE LEAF CONSTRUCTION

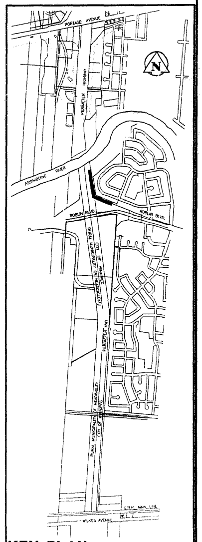






NOTE.
- 75mm layer of blown in place expanded urethane insulation on manhole walls only
- Flanged end alignment are ANSI B16 class 125

AIR RELEASE VALVE CHAMBER
AT STATION 34+80.0



KEY PLAN

CONSTRUCTION COMPLETION DATE - AUGUST 2001

REVISED AS CONSTRUCTED
BY V.M. DATE 2001/04/04
CHECKED BY R.M.

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

NOTE:
CHAINAGES CALCULATED ALONG CENTERLINE OF 400 PVC FM

NOTE:
ALL 400 FM BENDS RESTRAINED USING A SERIES 1300 UNI-FLANGE BLOCK BUSTER RESTRAINT DEVICE AND A THRUST BLOCK (SD-004,SD-005)

Bench Mark
81RG28 – Perimeter Hwy. (P.T.H. 100) & Roblin Blvd. Interchange; Tblt. on top of N. span (sidewalk) of overpass at interchange, 6.3m W. & 0.9m Nly. of N.E. Conc. edge of sidewalk on overpass.

EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PROFILE	PROPOSED
150 WM	WATERMAIN	150 WM	— — —	HYDRO	— — —	150 WM	WATERMAIN	150 WM
⌖	HYDRANT	✦	— —	M.T.S.	—·· —	± , ⊠	HYDRANT, VALVE	+ ⊠
⊗	VALVE	⊗	— — —	CONCRETE	═══	300 LDS	LAND DRAINAGE SEWER	300 LDS
300 LDS	LAND DRAINAGE SEWER	300 LDS	═══	ASPHALT	═══	250 WWS	WASTE WATER SEWER	250 WWS
250 WWS	WASTE WATER SEWER	250 WWS	═══	SIDEWALK	═══	×	℄ PROFILE	— — —
○	MANHOLE	●	———	PROPERTY LINE	— — —	⊡	NORTH OR WEST GUTTER	———
□	CATCH BASIN	■	⌖	SURVEY BAR	✦	⊙	SOUTH OR EAST GUTTER	— — —
▽	CURB INLET	▼	——▷	DITCH	——▶	◇	NORTH OR WEST ℞	
—+—+—	JUNCTIONS	—+—+—	31.334	ELEVATION	(32.231)	⊙	SOUTH OR EAST ℞	
▭	CULVERT	▭				△	NORTH OR WEST DITCH	- - - -
— —	GAS	—··—				▽	SOUTH OR EAST DITCH	—·—·—

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. 48-024
ELEV. 244.602m

NO.	REVISIONS	DATE	BY
2	REVISED AS CONSTRUCTED	2001.04.04	V.M.
1	ADDED WATER & WASTE COMMENTS	00.10.18	T.G.

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 F 204-453-9012

DESIGNED BY	C.D.	CHECKED BY	P.J.S.
DRAWN BY	T.G.	APPROVED BY	D.M.
HOR. SCALE:	1:1000	RELEASED FOR CONSTRUCTION:	
VERTICAL:	1:100		
DATE	00.03.06	DATE	

ENGINEER'S SEAL
10/25/00
C. L. DYCK

CONSULTANT DRAWING NO
PW-91-549

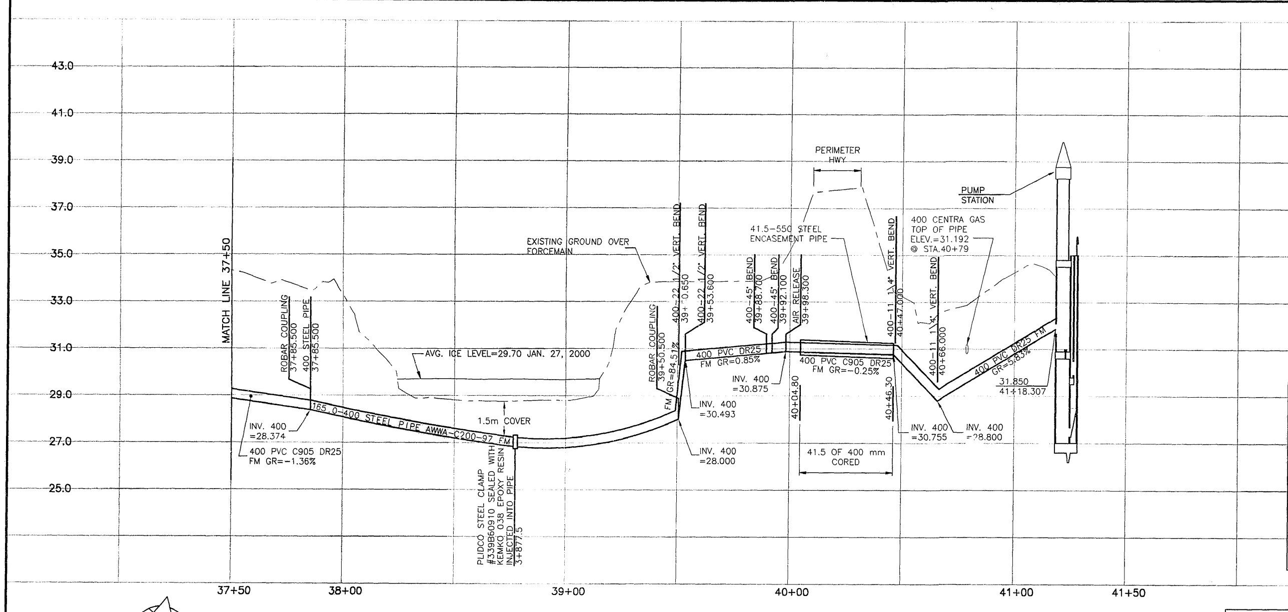
THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PERIMETER WEST FORCEMAIN
WASTE WATER SEWER AND PUMP STATION

400 FORCEMAIN
PHASE 2
STA 33+00 TO STA 37+50

DRAWING NUMBER
04787
SHEET 9 OF 24

W:\168\active\16802530 - Perimeter west 400 FM\Acad\temporary\04787.dwg
2002-08-09 10:36AM By: rmorris



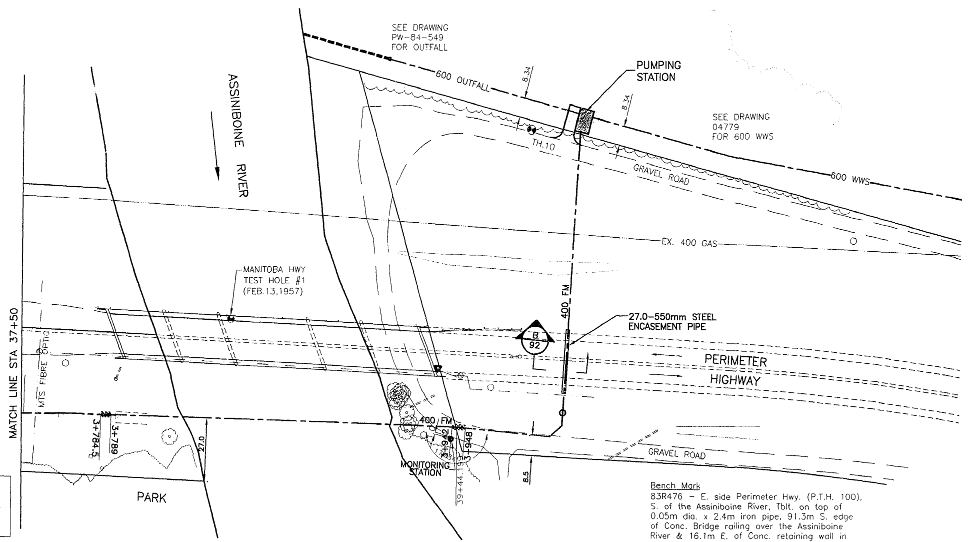
MATERIAL LIST

WASTE WATER SEWER	MANUFACTURER	SPECIFICATION	SPECIAL FEATURES
MANHOLE BASES	LAFARGE	ASTM C76	REINF. CONC.
MANHOLE RISERS	LAFARGE	ASTM C76	REINF. CONC.
FRAMES AND COVERS	TITAN FOUNDRY	CW 2131/CW 3205	C.I.
600 SEWER	IPEX	ASTM F769	PVC
600 OUTFALL SEWER	LAFARGE	ASTM C76	REINF. CONC.
400 FORCEMAIN PIPE	IPEX CENTURION	ASTM C-900	PVC
550 ENCASEMENT PIPE	CCTF (OAKVILLE, ONT.)	8.74mm THICK	STEEL
400 ENCASEMENT PIPE (RIVER CROSSING)	CCTF	9.53mm THICK	STEEL (DEV-TAR COATING)
600 CSP	ARMCO	2mm THICK	GALVANIZED METAL
600 SLIP JOINT	ARMCO		GALVANIZED METAL
400-45° BEND	GRINNELL	AWWA C-110	C.I.
400-22.5° BEND	IPEX	ASTM-C900	PVC
400 GATE VALVE	CLOW	AWWA C-509	
400-300 REDUCER	STAR	C-110	PVC
400-11.25° BEND	STAR	C-110	PVC
COUPLING	FORD		
400 COUPLING	ROBAR	C-110 DI	STEEL

CONTRACTOR: MAPLE LEAF CONSTRUCTION

FORCEMAIN TO BE SUPPORTED BY 4-38x89 STRAPPED LUMBER AS SHOWN THROUGH FULL LENGTH OF STEEL CASING PIPE

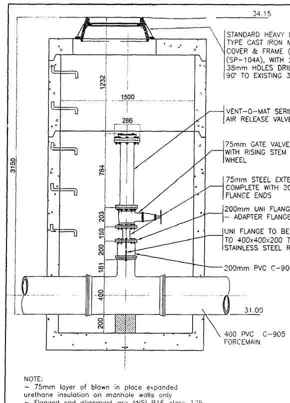
A 90 92 B 92 SECTION N.T.S.



METRIC

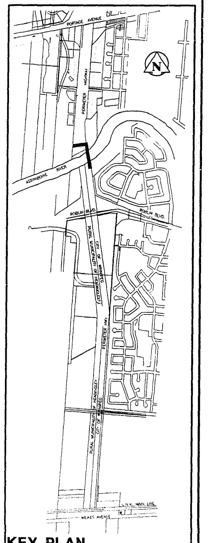
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

REVISED AS CONSTRUCTED
BY V.M. DATE 2001/04/04
CHECKED BY R.M.



NOTE:
- 75mm layer of blown in place expanded urethane insulation on manhole walls only
- Flanged end alignment are ANSI B16 class 125

AIR RELEASE VALVE CHAMBER
AT STATION 39+98.300



KEY PLAN

Bench Mark
83R476 - E. side Perimeter Hwy. (P.T.H. 100), S. of the Assiniboine River, Tblt. on top of 0.05m dia. x 2.4m iron pipe, 91.3m S. edge of Conc. Bridge railing over the Assiniboine River & 16.1m E. of Conc. retaining wall in center of Perimeter Hwy.

NOTE:
ALL 400 FM BENDS RESTRAINED USING A SERIES 1300 UNI-FLANGE BLOCK BUSTER RESTRAINT DEVICE AND A THRUST BLOCK (SD-004,SD-005)

NOTE:
CHAINAGES CALCULATED ALONG CENTERLINE OF 400 PVC FM

CONSTRUCTION COMPLETION DATE - AUGUST 2001

EXISTING	LEGEND-PLAN	PROPOSED
150 WM	WATERMAIN	150 WM
⌖	HYDRANT	✦
⊘	VALVE	⊗
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
○	MANHOLE	●
□	CATCH BASIN	■
▽	CURB INLET	▼
—+—+—	JUNCTIONS	—+—+—
▭	CULVERT	▭
— —	GAS	—··—

EXISTING	LEGEND-PLAN	PROPOSED
— — —	HYDRO	— — —
— —	M.T.S.	—·—
═	CONCRETE	═
═	ASPHALT	═
═	SIDEWALK	═
——	PROPERTY LINE	——
⌖	SURVEY BAR	✦
→	DITCH	→
31.334	ELEVATION	(32.231)

EXISTING	LEGEND-PROFILE	PROPOSED
150 WM	WATERMAIN	150 WM
±, X	HYDRANT, VALVE	±, X
300 LDS	LAND DRAINAGE SEWER	300 LDS
250 WWS	WASTE WATER SEWER	250 WWS
×	℄ PROFILE	— — —
⊡	NORTH OR WEST GUTTER	——
⊙	SOUTH OR EAST GUTTER	— — —
◇	NORTH OR WEST ℙ	
⬡	SOUTH OR EAST ℙ	
△	NORTH OR WEST DITCH	—·—·—
▽	SOUTH OR EAST DITCH	—··—··—

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. 48-025
ELEV. 237.037m

NO.	REVISIONS	DATE	BY
2	REVISED AS CONSTRUCTED	2001.04.04	V.M.
1	ADDED WATER & WASTE COMMENTS	00.10.18	T.G.

Stantec Consulting Ltd.
905 Waverley Street, Winnipeg, Manitoba
Tel 204-489-5900 Fax 204-453-9012
Stantec

DESIGNED BY C.D.
CHECKED BY F.J.S.
DRAWN BY T.G.
APPROVED BY D.M.
HOR. SCALE: 1:1000
VERTICAL: 1:100
RELEASED FOR CONSTRUCTION
DATE 00.03.06
DATE

ENGINEER'S SEAL
10/25/02
C. L. DYCK

CONSULTANT DRAWING NO.
PW-92-549

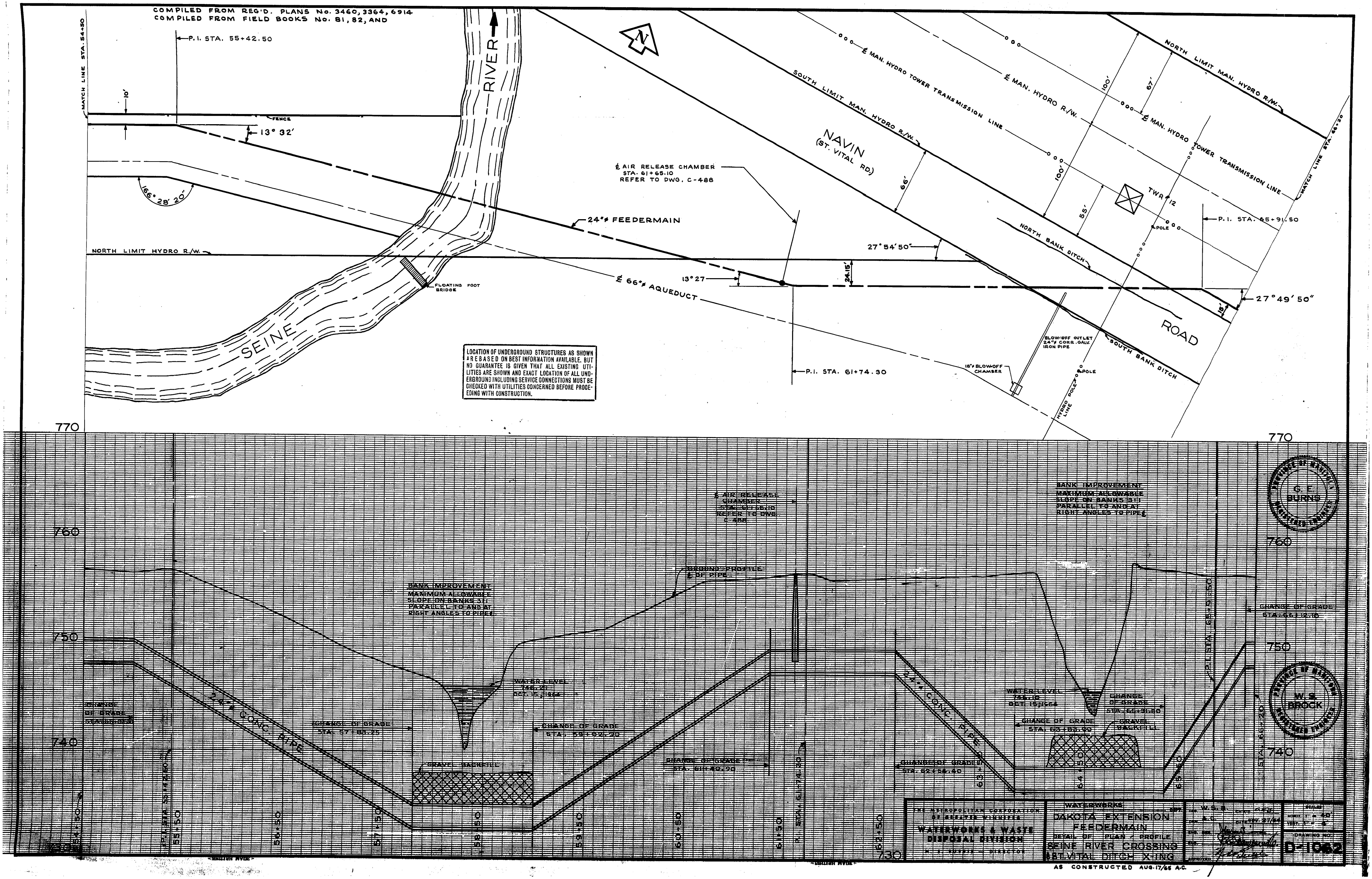
THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

PERIMETER WEST FORCEMAIN
WASTE WATER SEWER AND PUMP STATION
400 FORCEMAIN AND PUMP STATION
PHASE 2
STA 37+50 TO STA 41+50

CITY DRAWING NUMBER
04788
SHEET 10 OF 24

w:\168\active\16802530 - Perimeter west 400 FM\Acad\temporary\04788.dwg
2002-08-09 10:38AM By: rmorris

D-1062



THE METROPOLITAN CORPORATION OF GREATER WINNIPEG
WATERWORKS & WASTE DISPOSAL DIVISION
N. S. BUBBIS — DIRECTOR

WATERWORKS DEPT.
DAKOTA EXTENSION
FEEDERMAIN
DETAIL OF PLAN / PROFILE
SEINE RIVER CROSSING
& ST. VITAL DITCH X-ING

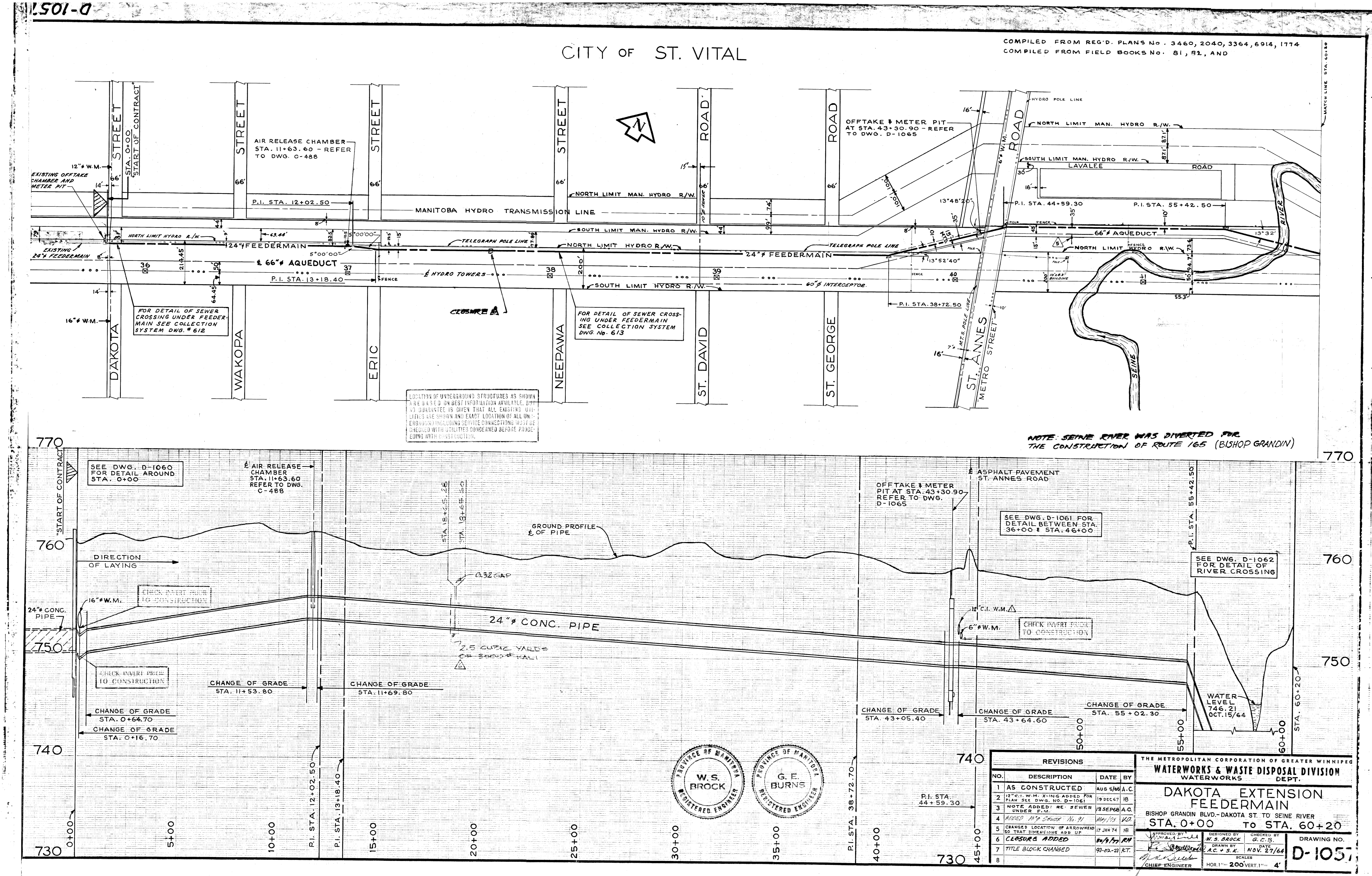
DES. W. S. B. CH'K'D. C.E.B.
DRN. A. C. DATE NOV. 27/64
ENG. DES.
ENG.
APPROVED

SCALES
HORIZ. 1" = 40'
VERT. 1" = 4'

DRAWING NO.
D-1062

AS CONSTRUCTED AUG. 17/65 A.C.

D-1057



COMPILED FROM REG'D. PLANS No. 3460, 2040, 3364, 6914, 1774
COMPILED FROM FIELD BOOKS No. 81, 82, AND

LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON BEST INFORMATION AVAILABLE, BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN AND EXACT LOCATION OF ALL UNDERGROUND INCLUDING SERVICE CONNECTIONS MUST BE CHECKED WITH UTILITIES CONCERNED BEFORE PROCEEDING WITH CONSTRUCTION.

NOTE: SEINE RIVER WAS DIVERTED FOR THE CONSTRUCTION OF ROUTE 165 (BISHOP GRANDIN)

W.S. BROCK

G.E. BURNS

REVISIONS

NO.	DESCRIPTION	DATE	BY
1	AS CONSTRUCTED	AUG 5/65	A.C.
2	12" C.I. W.M. X-ING ADDED FOR PLAN SEE DWG. NO. D-1061	19 DEC 67	HB
3	NOTE ADDED: RE: SEWER UNDER F.M.	13 SEP 68	A.C.
4	ADDED 10"ϕ SEWER No. 91	May/73	V.D.
5	CHANGED LOCATION OF ARROWHEAD SO THAT DIMENSIONS ADD UP	17 JAN 74	HB
6	CLOSURE ADDED	30/9/77	RH
7	TITLE BLOCK CHANGED	93-02-23	K.T.
8			

THE METROPOLITAN CORPORATION OF GREATER WINNIPEG
WATERWORKS & WASTE DISPOSAL DIVISION
WATERWORKS DEPT.

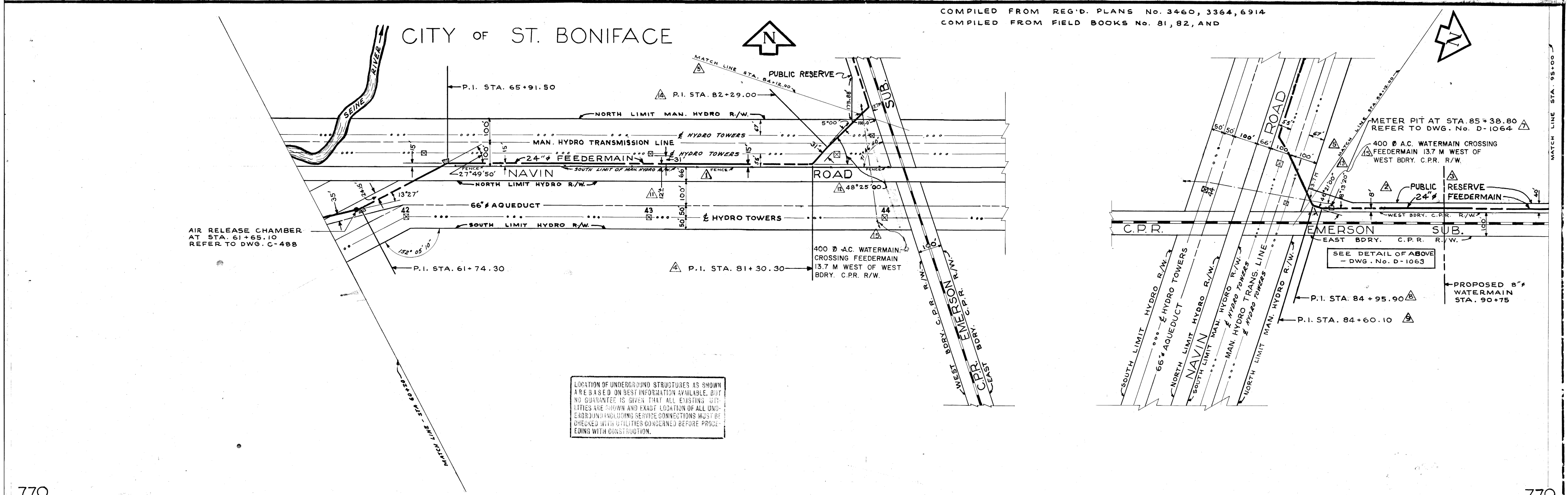
DAKOTA EXTENSION FEEDERMAIN

BISHOP GRANDIN BLVD.-DAKOTA ST. TO SEINE RIVER
STA. 0+00 TO STA. 60+20

APPROVED BY
CHIEF ENGINEER

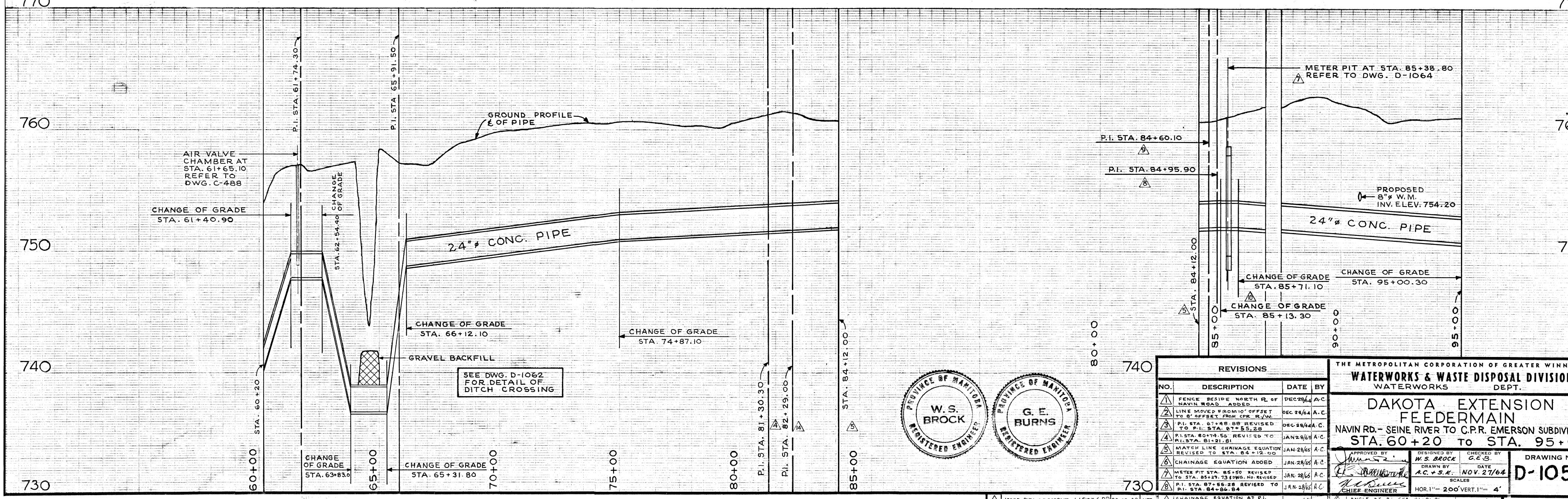
DESIGNED BY	W. S. BROCK
CHECKED BY	G. E. B.
DRAWN BY	A.C. + S.K.
DATE	NOV. 27/64
SCALES	HOR. 1" = 200' VERT. 1" = 4'
DRAWING NO.	D-1057

COMPILED FROM REG'D. PLANS No. 3460, 3364, 6914
COMPILED FROM FIELD BOOKS No. 81, 82, AND



SEE DETAIL OF ABOVE
- DWG. No. D-1063

LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON BEST INFORMATION AVAILABLE, BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN AND EXACT LOCATION OF ALL UNDERGROUND INCLUDING SERVICE CONNECTIONS MUST BE CHECKED WITH UTILITIES CONCERNED BEFORE PROCEEDING WITH CONSTRUCTION.



W. S. BROCK
G. E. BURNS

REVISIONS			
NO.	DESCRIPTION	DATE	BY
1	FENCE BESIDE NORTH ℞ OF NAVIN ROAD ADDED	DEC 28/64	A.C.
2	LINE MOVED FROM 10' OFFSET TO 8' OFFSET FROM CPR R./W.	DEC. 28/64	A.C.
3	P.I. STA. 87+48.88 REVISED TO P.I. STA. 87+55.28	DEC. 28/64	A.C.
4	P.I. STA. 80+74.56 REVISED TO P.I. STA. 81+21.81	JAN 28/65	A.C.
5	MATCH LINE CHAINAGE EQUATION REVISED TO STA. 84+12.00	JAN. 28/65	A.C.
6	CHAINAGE EQUATION ADDED	JAN. 28/65	A.C.
7	METER PIT STA. 85+50 REVISED TO STA. 85+29.73 & DWG. No. REVISED	JAN. 28/65	A.C.
8	P.I. STA. 87+55.28 REVISED TO P.I. STA. 84+86.84	JAN. 28/65	A.C.
9	CHAINAGE EQUATION AT P.I. REMOVED. REVISED = STA. 84+51.05	JAN. 28/65	A.C.
10	CHANGES OF GRADE STA. 85+30 & STA. 86+00 REVISED TO STA. 85+00 & STA 85+75	JAN. 28/66	A.C.
11	OFFSET OF 24"ϕ FEEDERMAIN REVISED TO CLEAR TOWER	MAR 2/65	A.C.
12	ANGLE AT P.I. STA. 81+21.81 REVISED FROM 45° TO 48°25'	MAR. 2/65	A.C.
13	ANGLE AT P.I. STA. 84+51.05 REVISED FROM 45° TO 46°21'	MAR. 2/65	A.C.
14	P.I. STA. 82+20.52 AT 5°00' ADDED.	MAR. 2/65	A.C.
15	ADDED PUBLIC RESERVE - W. SIDE C.P.R	93.02.23	KT

THE METROPOLITAN CORPORATION OF GREATER WINNIPEG
WATERWORKS & WASTE DISPOSAL DIVISION
WATERWORKS DEPT.

DAKOTA EXTENSION FEEDERMAIN

NAVIN RD. - SEINE RIVER TO C.P.R. EMERSON SUBDIVISION
STA. 60+20 TO STA. 95+00

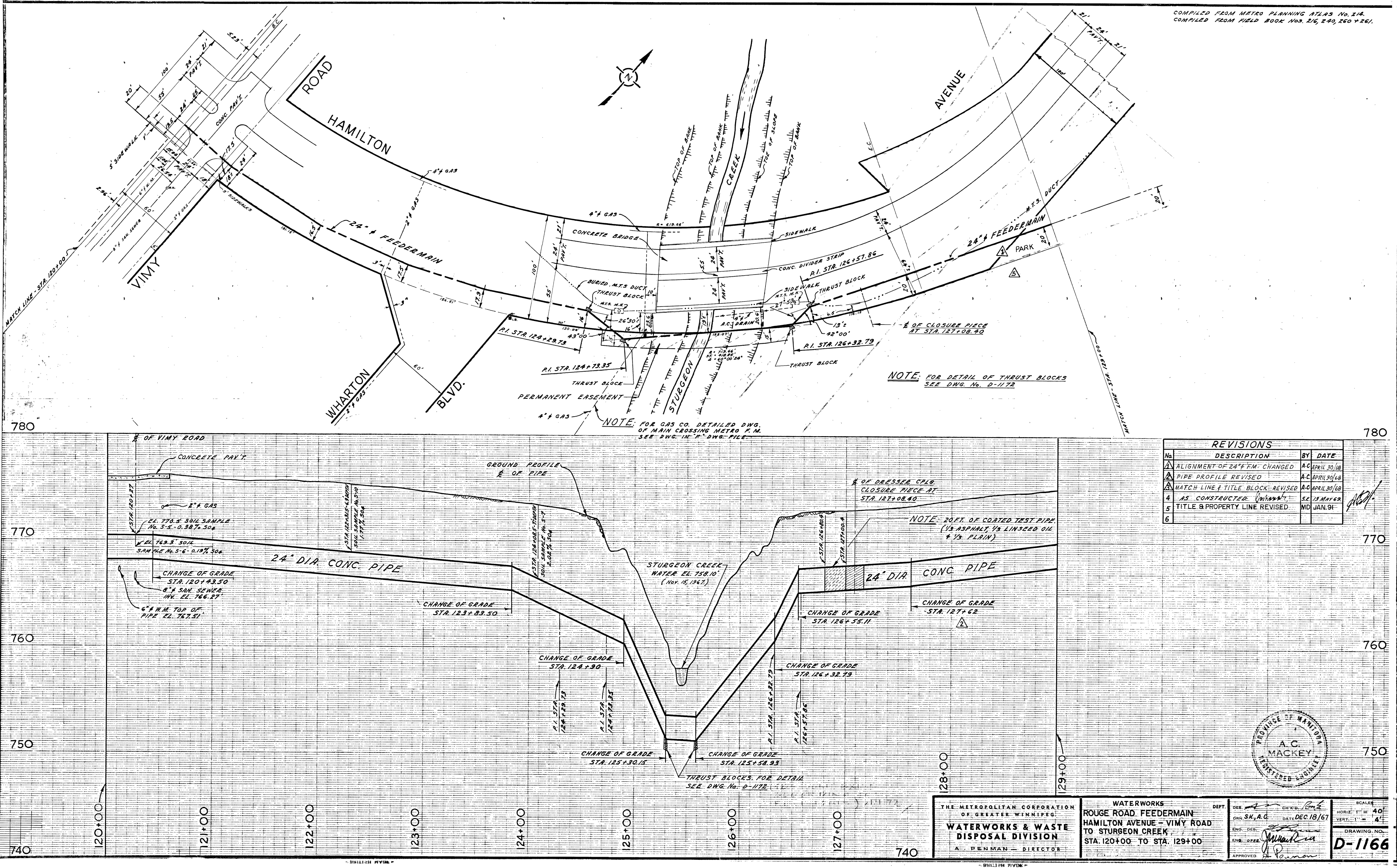
APPROVED BY
CHIEF ENGINEER
DESIGNED BY W. S. BROCK
CHECKED BY G.E.B.
DRAWN BY A.C. + S.K.
DATE NOV. 27/64
SCALES HOR. 1" = 200' VERT. 1" = 4'
DRAWING NO. D-1058

AS CONSTRUCTED AUG. 6/65 A.C.
400 wm X-ing (15) Jan. 86 mg

D-1058

D-1166

COMPILED FROM METRO PLANNING ATLAS No. 214.
COMPILED FROM FIELD BOOK Nos. 216, 240, 260 + 261.



NOTE: FOR GAS CO. DETAILED DWG. OF MAIN CROSSING METRO F.M. SEE DWG. IN "F" DWG. FILE.

NOTE: FOR DETAIL OF THRUST BLOCKS SEE DWG. No. D-1172

NOTE: 20 FT. OF COATED TEST PIPE. (1/3 ASPHALT, 1/3 LINSEED OIL & 1/3 PLAIN)

REVISIONS

No.	DESCRIPTION	BY	DATE
1	ALIGNMENT OF 24"ϕ F.M. CHANGED	A.C.	APRIL 30/68
2	PIPE PROFILE REVISED	A.C.	APRIL 30/68
3	MATCH LINE & TITLE BLOCK REVISED	A.C.	APRIL 30/68
4	AS CONSTRUCTED.	S.K.	13 MAY 69.
5	TITLE & PROPERTY LINE REVISED	MD	JAN. 91
6			

A.C. MACKEY

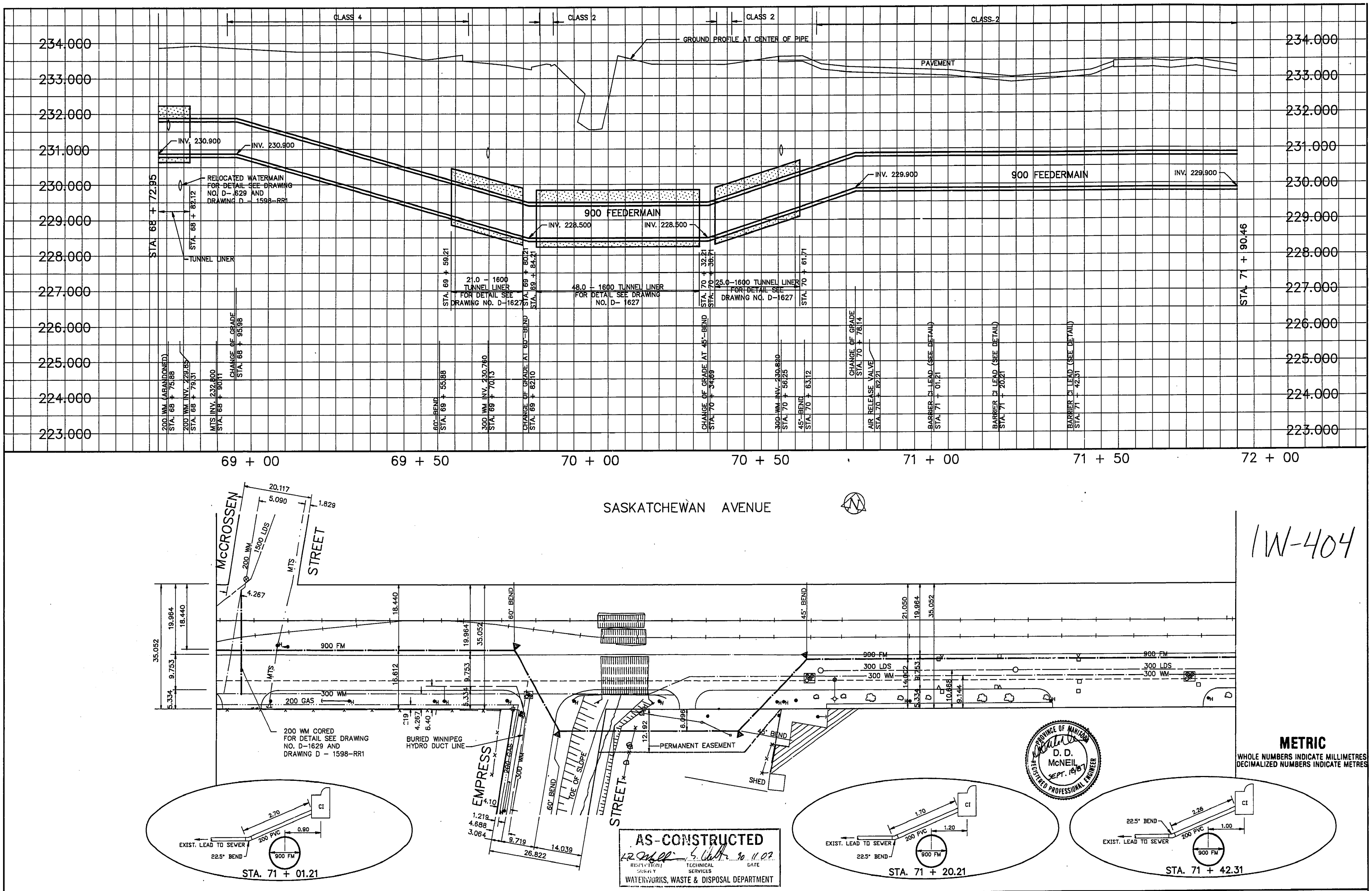
THE METROPOLITAN CORPORATION
OF GREATER WINNIPEG
WATERWORKS & WASTE
DISPOSAL DIVISION
A. PENMAN — DIRECTOR

WATERWORKS DEPT.
ROUGE ROAD FEEDERMAIN
HAMILTON AVENUE – VIMY ROAD
TO STURGEON CREEK
STA. 120+00 TO STA. 129+00

DES. CH'K'D.
DRN. S.K., A.C. DATE DEC. 18/67
ENG. DES.
ENG. OPER.
APPROVED A. Penman

SCALES
HORIZ. 1" = 40'
VERT. 1" = 4'

DRAWING NO.
D-1166



1W-404

D. D. McNEIL
Sept. 18/87

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

AS-CONSTRUCTED
L.R. M[illegible] S. [illegible] 90 11 07
INSPECTION/SURVEY TECHNICAL SERVICES DATE
WATERWORKS, WASTE & DISPOSAL DEPARTMENT

EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PLAN	PROPOSED	EXISTING	LEGEND-PROFILE	PROPOSED
200 WM	WATERMAIN	200 WM	— — —	SL , HYDRO		150 WM	WATERMAIN	150 WM
⌖	HYDRANT	◆	— · · —	MTS		┼	HYDRANT	┼
⊗	VALVE	●	═	CONCRETE	▨	X	VALVE	X
525 LDS	LAND DRAINAGE SEWER	525 LDS	——	PROPERTY LINE	— — — —	300 LDS	LAND DRAINAGE SEWER	300 LDS
375 WWS	WASTEWATER SEWER	375 WWS	⌖	SURVEY BAR		250 WWS	WASTEWATER SEWER	250 WWS
○	MANHOLE	●	—×—×—	FENCE		—×— · —	℄ PROFILE	—— · ——
□	CATCH BASIN	■	● ●H ●T	POLE – HYDRO, MTS				
▽	CURB INLET	▼	—●	RAILWAY SIGN				
┴┬┴	JUNCTIONS	┴┬┴	←.	GUY ANCHOR				
▭	CULVERT	▭	●—●L	LIGHT STANDARD				
50 GAS	GAS	50 GAS	✿	TREE				

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. ELEV.

NO.	REVISIONS	DATE	BY
1	REVISED TO AS-BUILT	89.09.28	TM

The City of Winnipeg
Works & Operations Division
Waterworks, Waste
& Disposal Department

DESIGNED BY RAS,DDM
CHECKED BY RAS,DDM
DRAWN BY REF,TM
APPROVED BY
HOR. SCALE 1:500
VERTICAL 1:50
RELEASED FOR CONSTRUCTION
DATE OCTOBER ,1986
DATE

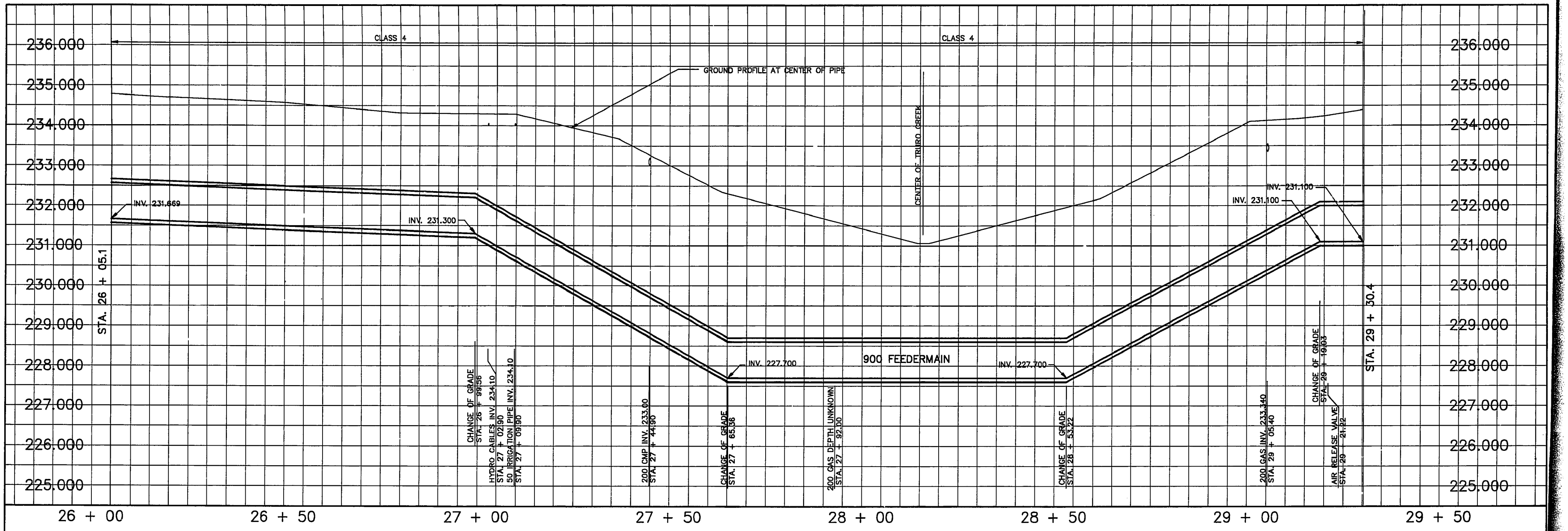
ENGINEER'S SEAL
R. A. SOROKOWSKI
Sept. 18/87

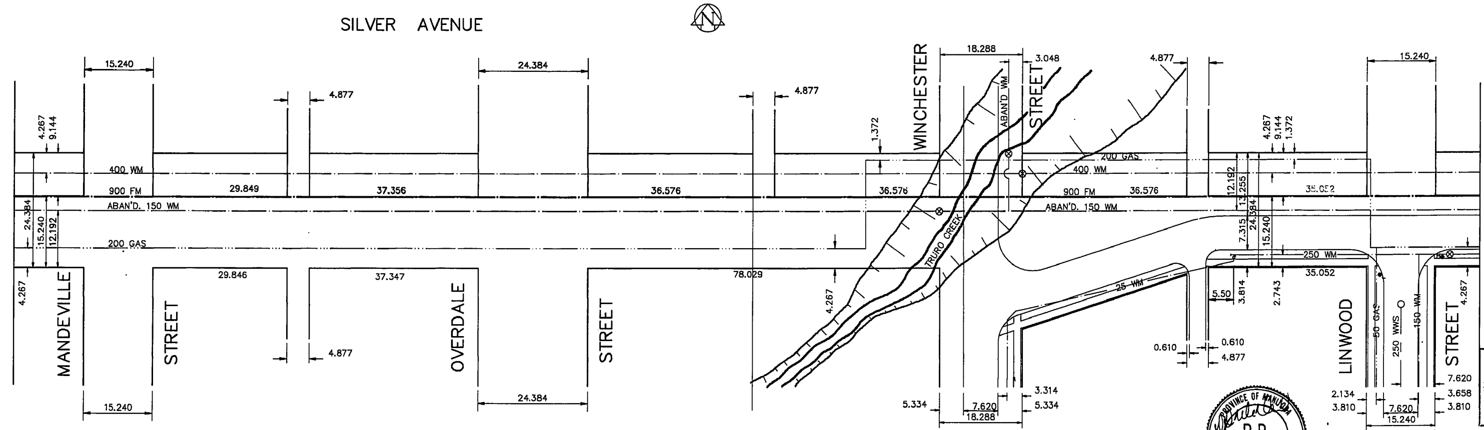
CONSULTANT DRAWING NO.

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION
WATERWORKS WASTE AND DISPOSAL DEPARTMENT

WEST END FEEDERMAIN
SASKATCHEWAN AVE. – McCROSSEN ST.
TO 270 METERS EAST
STA. 68 + 72.95 TO STA. 71 + 90.46

CITY DRAWING NUMBER
SHEET OF
D–1599





ZW-527

D. D. McNEIL SEPT. 18/87

AS-CONSTRUCTED

I.R.Mullin	S. Dalb	30.11.89
INSPECTION / SURVEY	TECHNICAL SERVICES	DATE

WATERWORKS, WASTE & DISPOSAL DEPARTMENT

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

200 WM	WATERMAIN	200 WM	— — —	SL , HYDRO		150 WM	WATERMAIN	150 WM
⌖	HYDRANT	✦	— · · —	MTS		— + —	HYDRANT	+
⊗	VALVE	●	═	CONCRETE	▨	— X —	VALVE	X
525 LDS	LAND DRAINAGE SEWER	525 LDS	——	PROPERTY LINE	— — — —	300 LDS	LAND DRAINAGE SEWER	300 LDS
375 WWS	WASTEWATER SEWER	375 WWS	⌖	SURVEY BAR		250 WWS	WASTEWATER SEWER	250 WWS
○	MANHOLE	●	✱—✱—✱	FENCE		—X·—	℄ PROFILE	—·—
□	CATCH BASIN	■	● ●H ●T	POLE – HYDRO, MTS				
▽	CURB INLET	▼	—●	RAILWAY SIGN				
┴┬┴	JUNCTIONS	┴┬┴	←	GUY ANCHOR				
▭	CULVERT	▭	—●L	LIGHT STANDARD				
50 GAS	GAS	50 GAS	✿	TREE				
EXISTING	LEGEND–PLAN	PROPOSED	EXISTING	LEGEND–PLAN	PROPOSED	EXISTING	LEGEND–PROFILE	PROPOSED

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE
DATE

NOTE:
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B.M.
ELEV.

NO.	REVISIONS	DATE	BY
1	REVISED TO AS-BUILT	89.04.10	TM

The City of Winnipeg
Works & Operations Division
Waterworks, Waste
& Disposal Department

DESIGNED BY RAS,DDM	CHECKED BY RAS,DDM
DRAWN BY REF,TM	APPROVED BY
HOR. SCALE 1:500 VERTICAL 1:50	RELEASED FOR CONSTRUCTION
DATE OCTOBER ,1986	DATE

ENGINEER'S SEAL
R. A. SOROKOWSKI SEPT. 18/87

CONSULTANT DRAWING NO.

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION
WATERWORKS WASTE AND DISPOSAL DEPARTMENT

WEST END FEEDERMAIN
SILVER AVE. – BELVIDERE ST. TO LINWOOD ST.
STA. 26 + 05.1 TO STA. 29 + 30.4

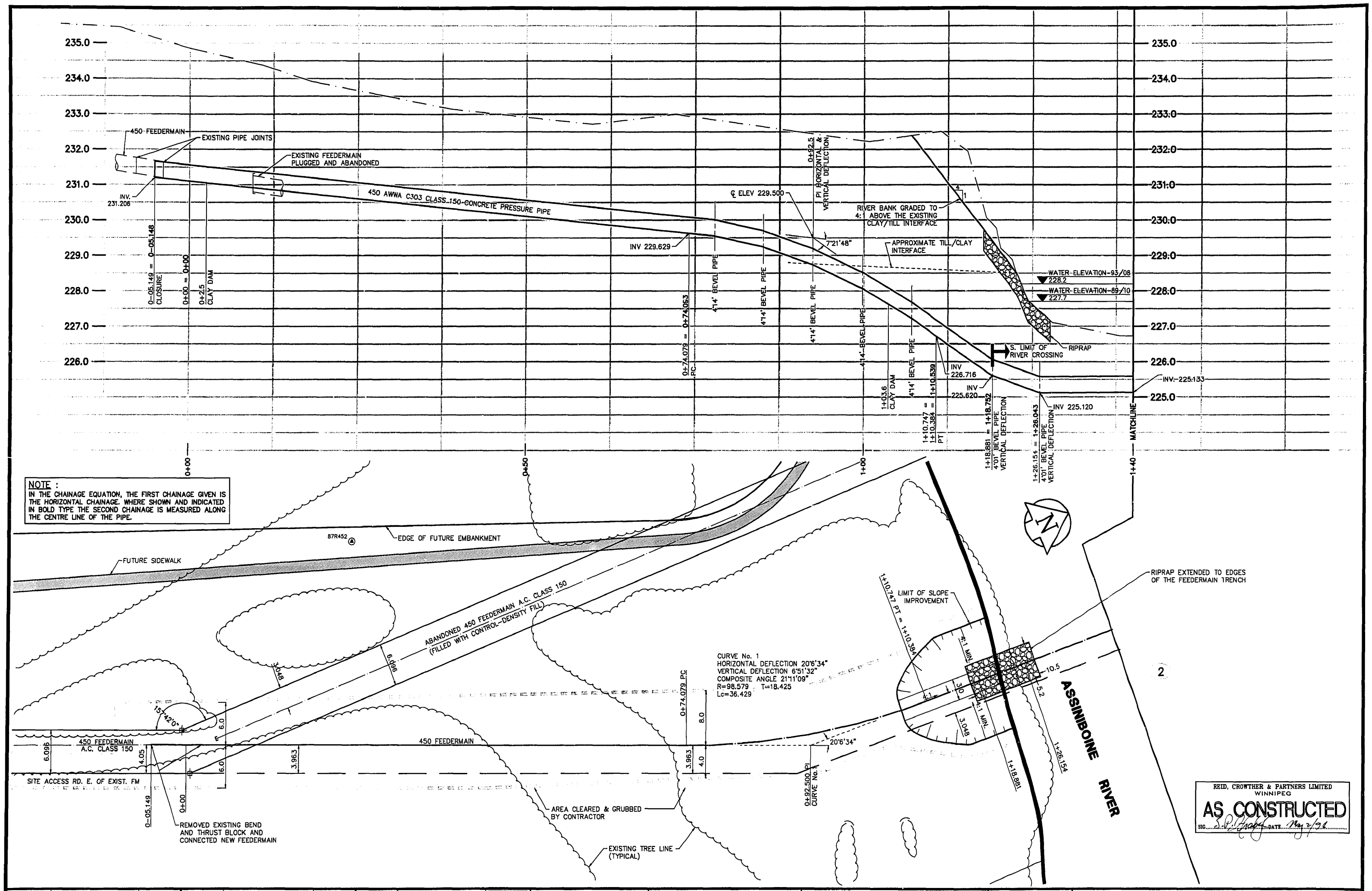
CITY DRAWING NUMBER
SHEET OF
D–1584

AZON CANADA 3130110300

REF. OR DRAWING NO. D-1955

DETAILS

File # FE10862



NOTE :
IN THE CHAINAGE EQUATION, THE FIRST CHAINAGE GIVEN IS THE HORIZONTAL CHAINAGE. WHERE SHOWN AND INDICATED IN BOLD TYPE THE SECOND CHAINAGE IS MEASURED ALONG THE CENTRE LINE OF THE PIPE.

REID, CROWTHER & PARTNERS LIMITED
WINNIPEG
AS CONSTRUCTED
SIG. S.P. Arab DATE May 2/94

EXISTING	LEGEND - PLAN	PROPOSED	EXISTING	LEGEND - PLAN	PROPOSED	EXISTING	LEGEND - PROFILE	PROPOSED
150 mm W.M.	WATERMAIN	150 mm W.M.	— - —	HYDRO	———	150 mm W.M.	WATERMAIN	150 mm W.M.
◇	HYDRANT	✦	———	M.T.S.	———	┼	HYDRANT	┼
⊗	VALVE	●	═	CONCRETE	▨	X	VALVE	X
300mm L.D.S.	LAND DRAINAGE SEWER	300mm L.D.S.	═	ASPHALT	▨	300mm L.D.S.	LAND DRAINAGE SEWER	300mm L.D.S.
250mm W.W.S.	WASTEWATER SEWER	250mm W.W.S.	═	PROPERTY LINE	- - - -	250mm W.W.S.	WASTE WATER SEWER	250mm W.W.S.
○	MANHOLE	●	⌖	SURVEY BAR		—×—	℄ PROFILE	—×—
□	CATCH BASIN	■	232.750	ELEVATION	(32.750)	150 mm F.M.	FEEDERMAIN	150 mm F.M.
▽	CURB INLET	▼	500ø	TREE				
—┼—┼—	JUNCTIONS	—┼—┼—	═	SIDEWALK RAMP	▬			
▭	CULVERT	▭	═	CONCRETE SIDEWALK	▨			
———	GAS	———	—×—×—	FENCE				

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE ________ DATE ________

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE, BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. 39-048 N. SIDE OF PORTAGE IN MEDIAN OF MORAY ST.
ELEV. 237.184 TBLT. ON TOP OF 0.05m x 2.4m IRON PIPE 0.5m N/N.L. PORTAGE & 12.8m W/E.L. MORAY

NO.	REVISIONS	DATE	BY
D	AS CONSTRUCTED	94.05.02	GSK
C	ISSUED FOR TENDER	93.09.15	DED
B	ISSUED FOR CITY REVIEW	93.08.24	GSK
A	ISSUED FOR APPROVAL	93.08.13	GSK

Reid Crowther
Reid Crowther & Partners Ltd.
Consulting Engineers

DESIGNED BY BAM/GSK
CHECKED BY BAM
DRAWN BY GSK
APPROVED BY
HOR. SCALE: 250
VERTICAL: 50
RELEASED FOR CONSTRUCTION:
DATE 93-06-28
DATE

ENGINEER'S SEAL
DESIGN ORIGINAL STAMPED BY B.A.MORTON AUG.16/93

CONSULTANT DRAWING NO.
61249-01-1

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION
WATER WORKS, WASTE AND DISPOSAL DEPT.

HANEY-MORAY FEEDERMAIN RELOCATION
298m NORTH OF ROBLIN BLVD. TO PINEWOOD DRIVE

FEEDERMAIN RELOCATION
0+00 TO 1+40

CITY DRAWING NUMBER
SHEET 1 OF 5
D-1955

File # FE10862

AZON CANADA 3130110300

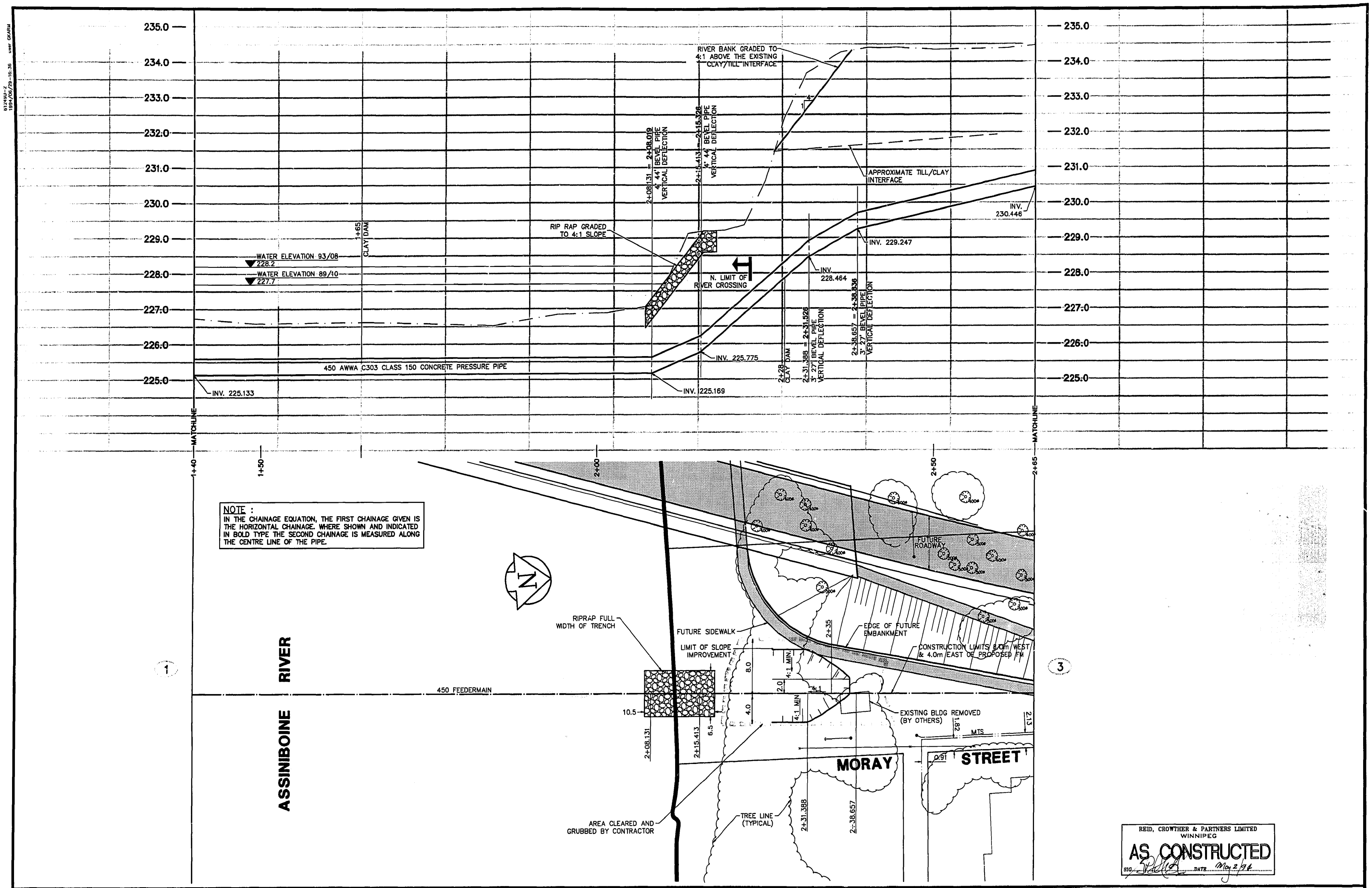
29x

SI Scale			
1:2500	50	100	150
1:250	5	10	15
1:25	.5	1.0	1.5

REF. or DRAWING No. D-1956

file # FE 10863

DETAILS



REID, CROWTHER & PARTNERS LIMITED
WINNIPEG
AS CONSTRUCTED
SIG. DATE May 2/94

EXISTING	LEGEND - PLAN	PROPOSED
150 mm W.M.	WATERMAIN	150 mm W.M.
◇	HYDRANT	✦
⊗	VALVE	●
300mm L.D.S.	LAND DRAINAGE SEWER	300mm L.D.S.
250mm W.W.S.	WASTEWATER SEWER	250mm W.W.S.
○	MANHOLE	●
□	CATCH BASIN	■
▽	CURB INLET	▼
	JUNCTIONS	
	CULVERT	
	GAS	

EXISTING	LEGEND - PLAN	PROPOSED
	HYDRO	
	M.T.S.	
	CONCRETE	▨
	ASPHALT	▨
	PROPERTY LINE	
⌖	SURVEY BAR	
232.750	ELEVATION	(32.750)
500ø	TREE	
	SIDEWALK RAMP	
	CONCRETE SIDEWALK	▨
	FENCE	

EXISTING	LEGEND - PROFILE	PROPOSED
150 mm W.M.	WATERMAIN	150 mm W.M.
+	HYDRANT	+
X	VALVE	X
300mm L.D.S.	LAND DRAINAGE SEWER	300mm L.D.S.
250mm W.W.S.	WASTE WATER SEWER	250mm W.W.S.
	℄ PROFILE	
150 mm F.M.	FEEDERMAIN	150 mm F.M.

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE DATE

NOTE:

LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE, BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M. 39-048
ELEV. 237.184
N. SIDE PORTAGE IN MEDIAN OF MORAY ST. TBLT. ON TOP OF 0.05m x 2.4m IRON PIPE. 0.5m N/N.L. PORTAGE & 12.8m W/E.L. MORAY

NO.	REVISIONS	DATE	BY
D	AS CONSTRUCTED	94.05.02	GSK
C	ISSUED FOR TENDER	93.09.15	DED
B	ISSUED FOR CITY REVIEW	93.08.24	GSK
A	ISSUED FOR APPROVAL	93.08.13	GSK

Reid Crowther
Reid Crowther & Partners Ltd.
Consulting Engineers

DESIGNED BY	BAM/GSK	CHECKED BY	BAM
DRAWN BY	GSK	APPROVED BY	
HOR. SCALE:	250	RELEASED FOR CONSTRUCTION:	
VERTICAL:	50		
DATE	93-06-28	DATE	

ENGINEER'S SEAL
DESIGN ORIGINAL STAMPED BY B.A.MORTON AUG.16/93

CONSULTANT DRAWING NO.
61249-01-2

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION
WATER WORKS, WASTE AND DISPOSAL DEPT.

HANEY-MORAY FEEDERMAIN RELOCATION
298m NORTH OF ROBLIN BLVD. TO PINEWOOD DRIVE
FEEDERMAIN RELOCATION
1+40 TO 2+65

CITY DRAWING NUMBER
SHEET 2 OF 5
D-1956

File # FE 10863

Appendix B

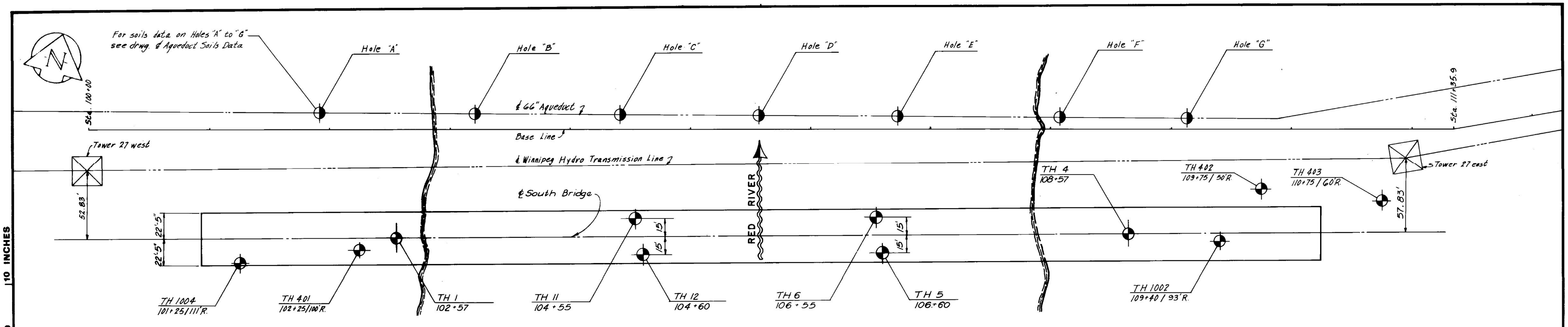
B1: Site 4 Existing Geotechnical Information

B2: Site 5 Existing Geotechnical Information

B3: Site 7 Existing Geotechnical Information

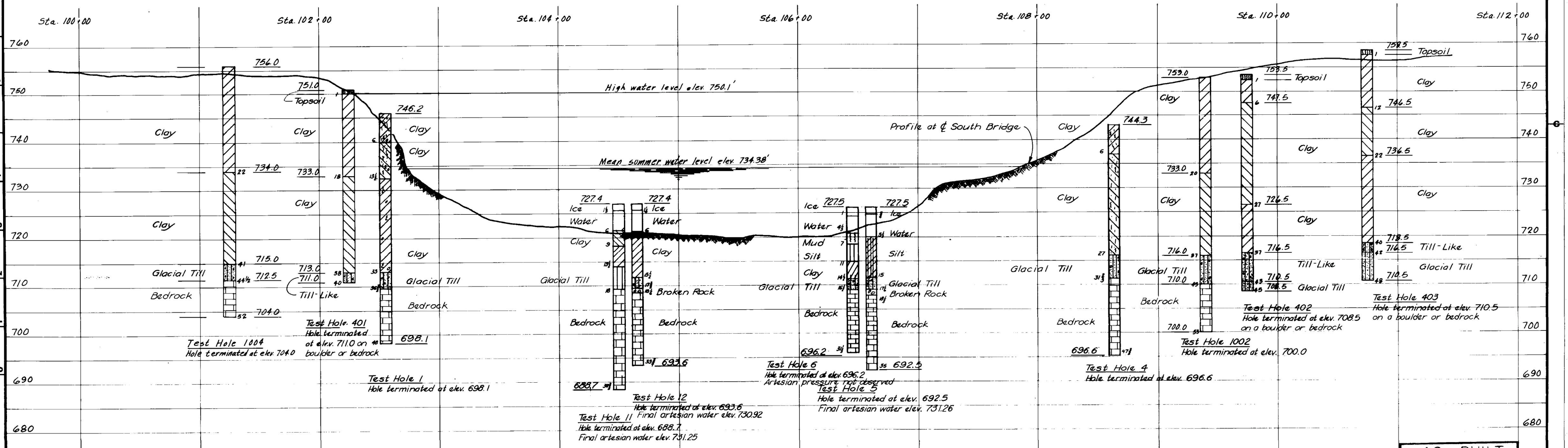
B4: Site 8 Existing Geotechnical Information

B5: Site 9 Existing Geotechnical Information



TEST HOLE LOCATION PLAN

Scale: 1"=40'-0"



TEST HOLE DATA

Scale: Horiz. 1"=40'-0"
Vert. 1"=10'-0"

AS-BUILT

DATE	F.B. NO.	PAGE
Nov. 16/79		

Legend:

◕ Bridge test holes

◑ Aqueduct test holes for information only, taken from existing data (1960) see aqueduct soils data.

Notes:

1. Subsurface information shown on this drawing was obtained solely for use in establishing design controls for the project. This information is not guaranteed to be accurate or all-inclusive and it is not to be construed as part of the plans governing construction of the project. The Contractor is to satisfy himself as to actual conditions prevailing at the site.
2. Water levels measured at James Avenue Pumping Station.
3. The above drawing should be read in conjunction with Klohn Leonoff Consultants Ltd's Report W-1064.

REVISIONS

D. L. MILLS

THE CITY OF WINNIPEG
WORKS & OPERATIONS DEPARTMENT
STREETS & TRANSPORTATION DIVISION

W. L. WARDROP & ASSOCIATES LTD.
ENGINEERING CONSULTANTS
WINNIPEG · THUNDER BAY · REGINA · BARRIE · EDMONTON

APPROVED BY: D.R. Grimes DATE 25 Mar/77

DRAWN BY LMG DATE
PRELIM. CHK. L.L.S. 14/1/77
DESIGN KWS DATE NOV 76
CHECK Jan 77

ROUTE 165

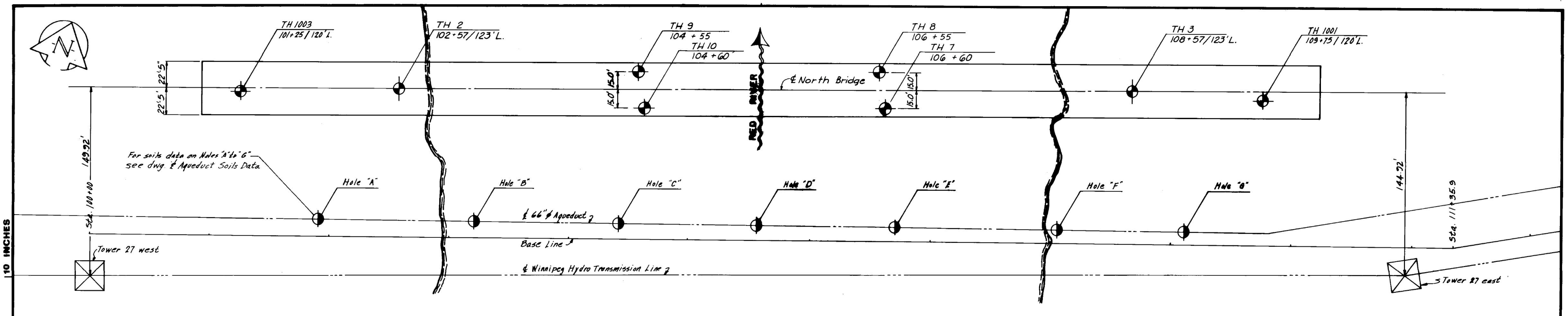
SOUTH BRIDGE SOILS DATA

APPROVED BY: Gerald R. Campbell 25/3/77
MANAGER OF STREETS AND TRAFFIC DATE

SCALE: AS SHOWN

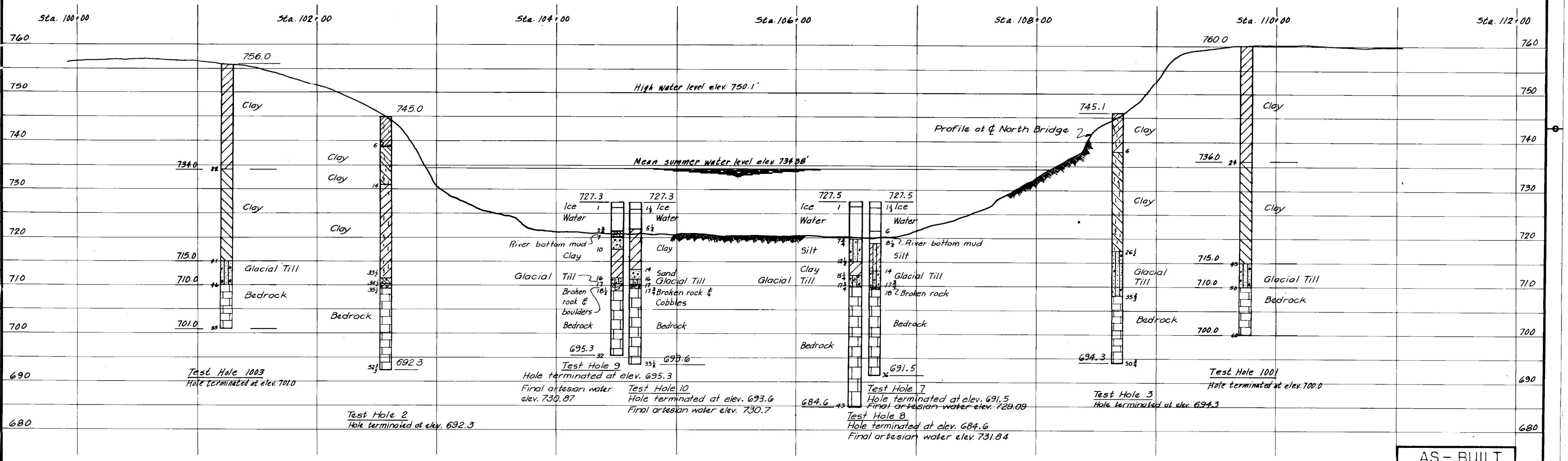
DRAWING NO. B-5092-205

W.L.W. NO. 74012-21



TEST HOLE LOCATION PLAN

Scale: 1"=40'-0"



TEST HOLE DATA

Scale: Horiz. 1"=40'-0"
Vert. 1"=10'-0"

Legend:

- Bridge test holes
- Aqueduct test holes for information only, taken from existing data (1960) see aqueduct soils data.

Notes:

1. Subsurface information shown on this drawing was obtained solely for use in establishing design controls for the project. This information is not guaranteed to be accurate or all-inclusive and it is not to be construed as part of the plans governing construction of the project. The Contractor is to satisfy himself as to actual condition prevailing at the site.
2. Water levels measured at James Avenue Pumping Station.
3. The above drawing should be read in conjunction with Klohn Leonoff Consultant Ltd's Report W-1064.

AS-BUILT

DATE	FB NO	PAGE
Nov. 16/79		

REVISIONS BY

D. L. MILLS

THE CITY OF WINNIPEG
WORKS & OPERATIONS DEPARTMENT
STREETS & TRANSPORTATION DIVISION

W. L. WARDROP & ASSOCIATES LTD.
ENGINEERING CONSULTANTS
WINNIPEG · THUNDER BAY · REGINA · BARRIE · EDMONTON

APPROVED BY: DATE 23 Mar/77

DRAWN BY LMG DATE
PRELIM. CHK. KWS Jan 77
DESIGN KWS DATE NOV.76
CHECK

ROUTE 165

NORTH BRIDGE SOILS DATA

SCALE: AS SHOWN

APPROVED BY: Lowell R Campbell 25/3/77
MANAGER OF STREETS AND TRAFFIC DATE

DRAWING NO. B-5092-206

W.L.W. NO. 74012-21

TEST HOLE LOG

SAMPLE DATA

WEIGHT HAMMER

HEIGHT DROP

ELEV. COLLAR Tech: S. Gilchrist

ELEV. GROUND 751

CO-ORD. LOCATION 101+ 25 - 100' Right

DEPTH / ELEV.	O.D / I.D.	BLOWS / FT	NO.
	3"Sy		1
10 / 741	3"Sy		2
	3"Sy		3
20 / 731	3"Sy		4
	3"Sy		5
30 / 721	3"Sy		6
	3"Sy		7
40 / 711	Bag		8

SYMBOL

DESCRIPTION OF MATERIAL

1 TOPSOIL

CLAY - silty
- medium plastic
- grey
- nuggetty to massive
- silt pockets to 1/8" diam
- stiff to very stiff

18

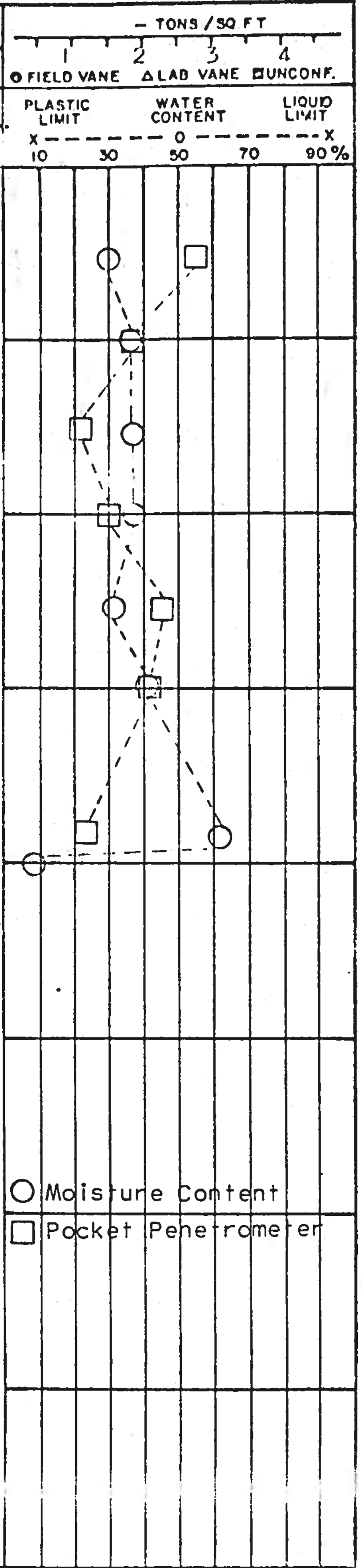
CLAY - highly plastic
- grey
- massive
- sand pockets <1/16" in diam to 20'
- silt pockets <1/16" in diam
- stiff to very stiff

38

40 TILL-LIKE - silty
- tan
- hard

NOTES:
1. Hole terminated at 40' on a boulder in till-like material.
2. About 1½' of water in hole when finished drilling.

Unconfined Compression



Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No. W00983

PROJECT Transportation Corridor

LOCATION Ft Garry/St Vital, Manitoba

HOLE No. 401

DATE Feb 13/76

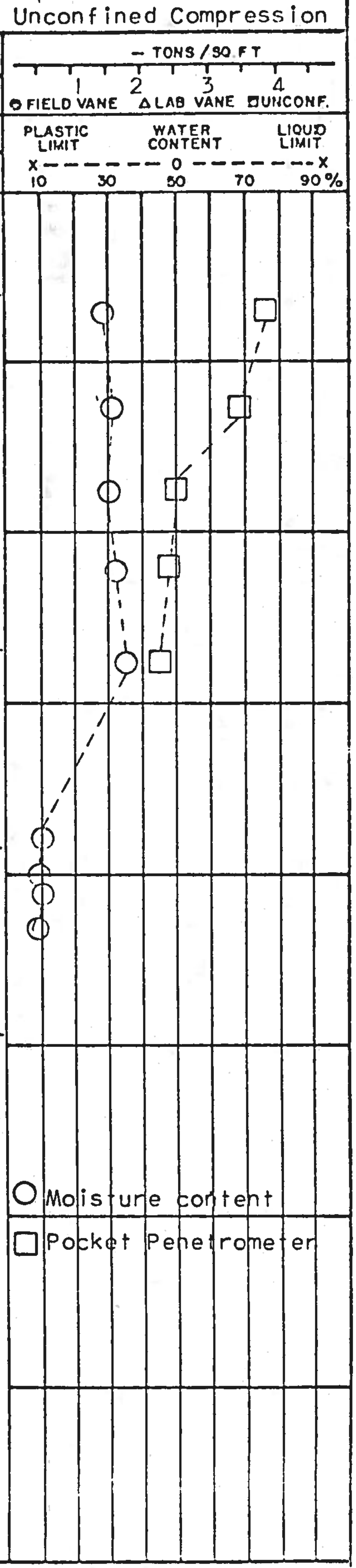
PLATE A-W-983-136

TEST HOLE LOG

SAMPLE DATA	
WEIGHT HAMMER	
HEIGHT DROP	
ELEV. COLLAR	Tech: S. Gilchrist
ELEV. GROUND	753.5
CO-ORD. LOCATION	109 + 75 – 50' Right

DEPTH ELEV	O.D. I.D.	BLOWS FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					1 TOPSOIL
					CLAY – silty, grey
	3"Sy		1		6 CLAY – medium plastic – brown – massive – silt pockets to ¼" in diam – very stiff
10					
743.5	3"Sy		2		
	3"Sy		3		
20					
733.5	3"Sy		4		
	3"Sy		5		27 CLAY – highly plastic – grey – massive – silt pockets to 1/8" in diam – sand pockets to ½" in diam – very stiff
30					
723.5					
	Bag		6		37 TILL-LIKE – silty – gravelly – tan – clay seams < 1/8" thick to 39' – soft to firm
40	Bag		7		
713.5			8		
	Bag		9		43 GLACIAL TILL – silty – tan – hard
					45
50					
703.5					

NOTES:
1. Hole terminated at 45' on a boulder in glacial till.
2. Slight seepage at 45' or elevation 708.5.



Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

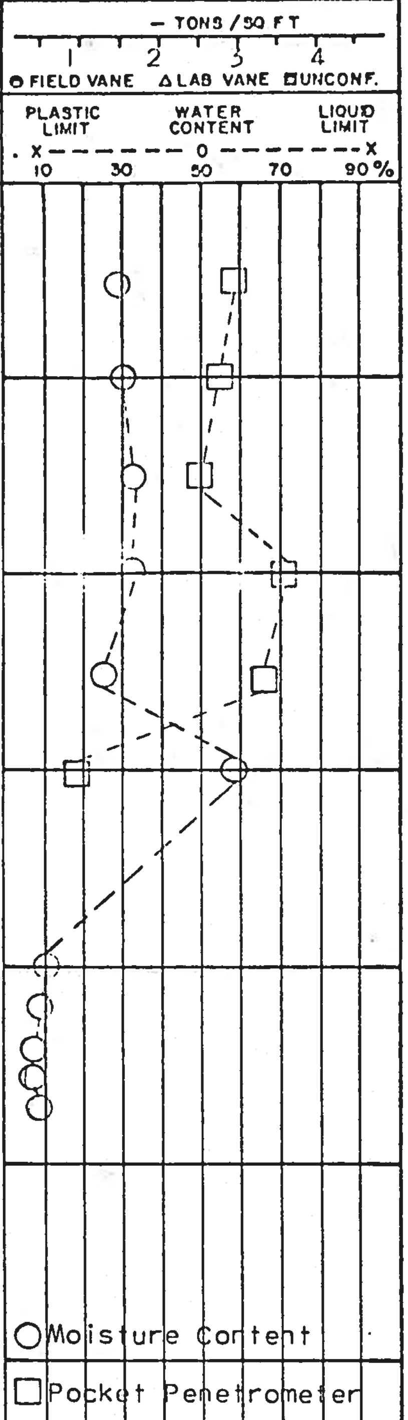
JOB No.	WG0085
PROJECT	Transportation Corridor
LOCATION	Ft Garry/St Vital, Manitoba
HOLE No.	402
DATE	Feb 16/75
PLATE	A-W-083-137

TEST HOLE LOG

Unconfined Compression

SAMPLE DATA	ELEV. COLLAR Tech: S. Gilchrist
WEIGHT HAMMER	ELEV. GROUND 758.5
HEIGHT DROP	CO-ORD. LOCATION 110 + 75. - 60' Right

DEPTH ELEV	O.D. I.D.	BLOWS FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					1 TOPSOIL
	3"Sy		1		CLAY - silty - medium plastic - brown - massive - very stiff
10 748.5	3"Sy		2		
					12
	3"Sy		3		CLAY - highly plastic - mottled brown - massive - small pebbles to 1/8" in diam - silt pockets to 1/16" in diam - very stiff
20 748.5	3"Sy		4		
					22
	3"Sy		5		CLAY - highly plastic - grey - massive - sand seams 1/16" thick to 25' - silt pockets to 3/8" in diam - small pebbles to ¼" in diam - firm to very stiff
30 728.5	3"Sy		6		
40 718.5			7		40
	Bag Bag Bag Bag Bag		8 9 10 11		42 TILL-LIKE - silty - sandy - gravelly - tan - soft
					48
50 708.5					GLACIAL TILL - silty - tan - hard to very hard



NOTES:

1. Hole terminated at 48' on a boulder in glacial till.
2. Water seepage from 47'.
3. About 6" of water in the bottom of the hole when drilling finished.

Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No.	WG0983
PROJECT	Transportation Corridor
LOCATION	Ft Garry/St Vital, Manitob
HOLE No.	403
DATE	Feb 16/75 PLATE A-W-983-138

TEST HOLE LOG

North Test hole
East Abutment

SAMPLE DATA	
WEIGHT HAMMER	140
HEIGHT DROP	30"

Tech: J. A. Odermatt
ELEV. GROUND 760
CO-ORD. LOCATION 109 + 75; 120' left

– TONS / SQ FT
1 2 3 4

PLASTIC LIMIT X 10 · 30 · WATER CONTENT O 50 · 70 · LIQUID LIMIT X 90 %

DEPTH / ELEV	O.D / I.D.	BLOWS / FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					CLAY – mottled brown
10					
750					
20					
740				24	CLAY COLOUR CHANGED TO GREY
30					
730					
40					
720				45	GLACIAL TILL – soft to very stiff – grey
50	S.S.	35	1	50	LIMESTONE – hard – tight horizontal parting – whitish to cream – no water loss – smooth drilling – 21% to 70% recovery
710					
60				60	
700					
70					
690					

NOTES:
1. Hole terminated at 60'.
2. "B" casing to 50', couple of inches into rock.
3. Water at 24' the next morning or elevation 736.
4. Ford's Mayhew rig.
5. Coring 52'-55'4" – 70% recovery.
6. Coring 55'4"-60' – 21% recovery.
7. Possibly weathered to 52'.

○ Moisture Content
☐ Pocket Penetrometer
Unconfined compression
Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No WG0083
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1001
DATE Jan 20/76 PLATE A-W-083-140

TEST HOLE LOG

South Test Hole
East Abutment

SAMPLE DATA
WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 753
CO-ORD. LOCATION 109 + 40; 93' right

— TONS / SQ FT 1 2 3 4

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X
10 30 50 70 90 %

DEPTH ELEV	O.D. I.D.	BLOWS FT	NO.	SYMBOL	DESCRIPTION OF MATERIAL
					CLAY – mottled brown
10 743					
20 733					20
					CLAY COLOUR CHANGED TO GREY
30 723					
					37
40 713	S.S.	30	1		GLACIAL TILL – soft to stiff 37-39' – stiff to very stiff 39-43'
					43
					LIMESTONE – sound – very hard – white crystaline – no water loss – 25% to 80% recovery
50 703					53
60 793					
70 783					

NOTES:
1. Hole terminated at 53'.
2. "B" casing to 43'.
3. No water loss in till and/or limestone.
4. Ford's Mayhew rig.
5. Coring 43'-48' – 25% recovery.
6. Coring 48'-53' – 80% recovery.
7. Possibly weathered rock 41'6" to 43'0".

○ Moisture Content
☐ Pocket Penetrometer Unconfined compression Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0983
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1002
DATE Jan 21/76 PLATE A-W-983-J41

TEST HOLE LOG

North Test Hole
West Abutment

SAMPLE DATA	
WEIGHT HAMMER	140
HEIGHT DROP	30"

Tech: J. A. Odermatt

ELEV. GROUND 756

CO-ORD. LOCATION 101 + 25; 120' left

DEPTH / ELEV
10
746
20
736
30
726
40
716
50
706
60
996
70
986

O.D / I.D	BLOWS / FT	NO.	SYMBOL	DEPTH	DESCRIPTION OF MATERIAL
			SS	2	SNOW (snow dump area)
			▨		CLAY – mottled brown
			▨	22	CLAY COLOUR CHANGED TO GREY
S.S. 30/2"			∘	41	GLACIAL TILL – firm to stiff – grey – till-like
			▭	46	LIMESTONE – cream to rust 46-50' with some silty clay – sound, very hard, white crystaline with light horizontal partings – 45% to 75% recovery
				55	

– TONS / SQ FT

1 2 3 4

PLASTIC LIMIT X — WATER CONTENT O — LIQUID LIMIT X

10 30 50 70 90%

NOTES:

1. Hole terminated at 55'.
2. Smooth drilling in rock.
3. No water loss.
4. Coring 46'-50'6" – 45% recovery.
5. Coring 50'6"-55' – 75% recovery.
6. Weathered rock 45'9" to 46'0".

○ Moisture Content

☐ Pocket Penetrometer Unconfined Compression Tons per sq ft.

Klohn Leonoff Consultants Ltd.

CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0085

PROJECT Transportation Corridor

LOCATION Ft Garry/St Vital, Manitoba

HOLE No. 1003

DATE Jan 22/76 PLATE A-W-085-142

TEST HOLE LOG

South Test Hole
West Abutment

SAMPLE DATA
WEIGHT HAMMER 140
HEIGHT DROP 30"

Tech: J. A. Odermatt
ELEV. GROUND 756
CO-ORD. LOCATION 101 + 25; 111' right

– TONS / SQ FT
1 2 3 4

PLASTIC LIMIT X – – – WATER CONTENT O – – – LIQUID LIMIT X
10 30 50 70 90 %

DEPTH / ELEV	O.D. / I.D.	BLOWS / FT	NO.		DESCRIPTION OF MATERIAL
					CLAY – mottled brown
10 746					
20 736				22	
					CLAY COLOUR CHANGED TO GREY
30 726					
40 716	S.S.	11/6"	1	41	GLACIAL TILL – firm to stiff – grey
	S.S.	30/½"	2	44½	LIMESTONE – sound, hard – brown to white – horizontal partings – 86% to 88% recovery
50 706				52	
60 996					
70 986					

NOTES:
1. Hole terminated at 52'.
2. Coring 44'6"-48' – 86% recovery.
3. Coring 48'-52' – 88% recovery.

○ Moisture Content

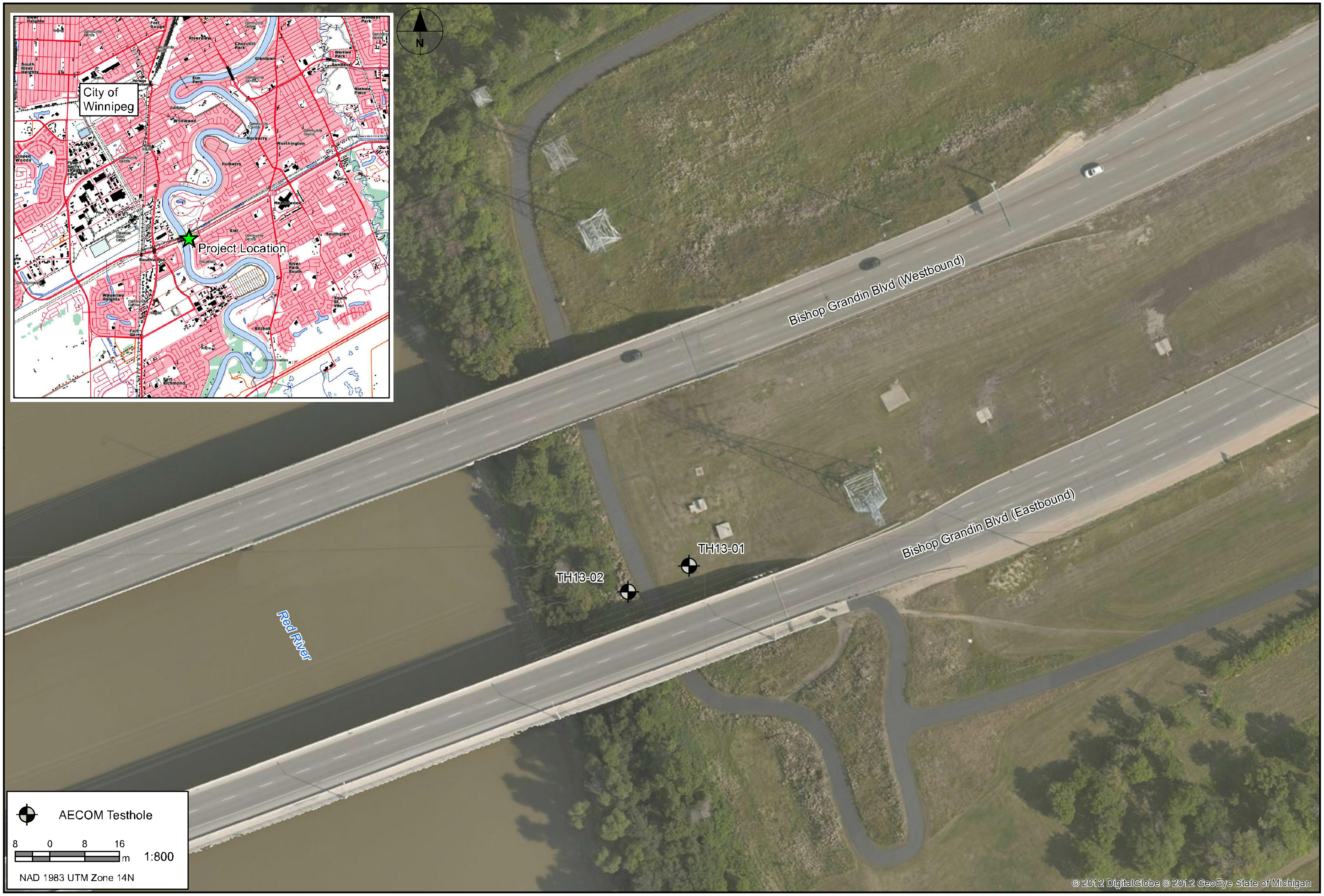
☐ Pocket Penetrometer
Unconfined Compression
Tons per sq ft.

Klohn Leonoff Consultants Ltd.
CIVIL & GEOTECHNICAL ENGINEERS

JOB No. WG0083
PROJECT Transportation Corridor
LOCATION Ft Garry/St Vital, Manitoba
HOLE No. 1004
DATE Jan 23/76 PLATE A-W-083-43

Project Management Initials: Designer: Checked: Approved: ANSI B 279.4mm x 431.8mm

Last saved by: MAHEC (2013-12-12) Last Plotted: 2013-12-12
Filename: P:\60274906\000-CADD\GIS\MXD\G001_60274906_01V1_TESTHOLELOCATIONS.MXD



AECOM

Figure: A1

The Repair of the Fort Garry Interceptor Sewer Crossing

Testhole Location Plan

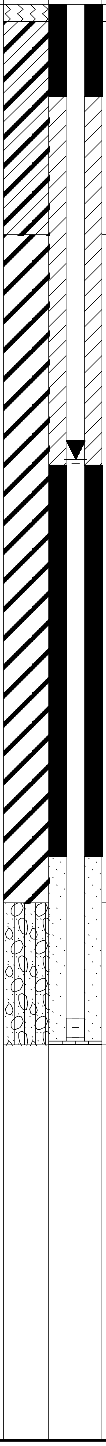
City of Winnipeg, Fort Garry Sewer Interceptor

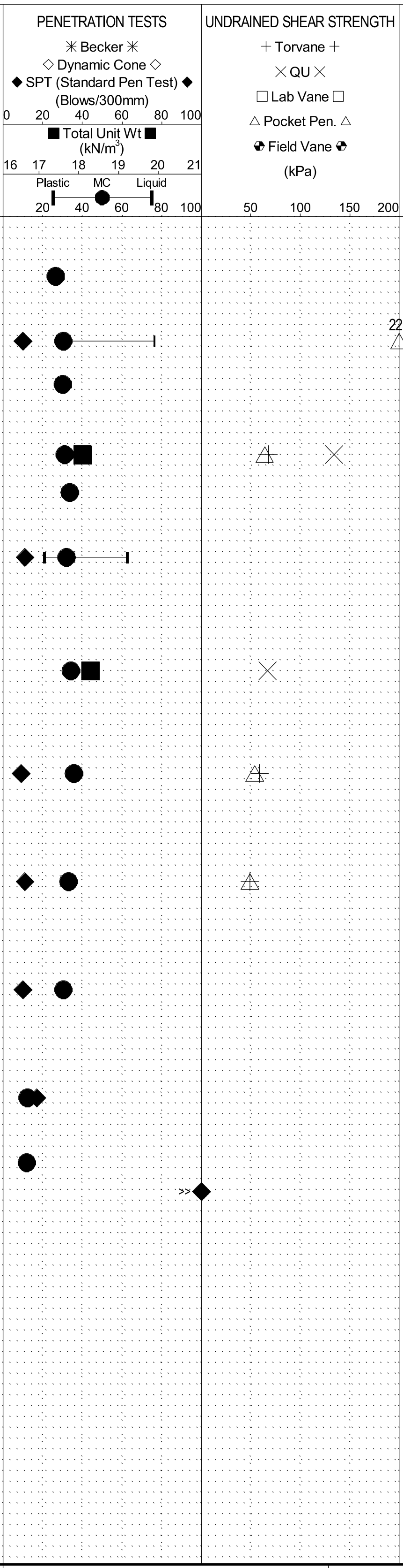
PROJECT: FGSV Interceptor Siphon	CLIENT: City of Winnipeg	TESTHOLE NO: **TH13-01**
LOCATION: Upper Bank of Red River, UTM: 14 U, N 5520496, E 0633705		PROJECT NO.: 60274906
CONTRACTOR: Paddock Drilling	METHOD: Truck Mounted Acker MP-8	ELEVATION (m):

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▧ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁙ GROUT ▨ CUTTINGS ⁘ SAND

DEPTH (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
0	TOPSOIL and ORGANICS - some clay - brown, dry				
1	CLAY- silty, trace sand, trace organics, trace sulphates - brown, dry, stiff - high plasticity - moderately fissured	■	G01		
2		☒	S02	10	SPT Blows: 4, 5, 5 40% Recovery Gravel: 0.0 %, Sand: 0.5%, Silt: 30.3%, Clay: 69.2%
		■	G03		
3	CLAY and SILT - trace sand - brown, stiff, dry to moist - high plasticity	▥	T04		(T04): 60% Recovery
4	- mottled brown grey below 4.1 m	■	G05		
5		☒	S06	11	SPT Blows: 5, 4, 7 100% Recovery Gravel: 0.0%, Sand: 1.4%, Silt: 47.8%, Clay: 50.8%
6		▥	T07		(TO7): 100% Recovery
7	- wet at 6.7 m				
8	- fine sand lense (25 mm thickness) at 7.8 m - grey below 7.8 m	☒	S08	9	SPT Blows: 3, 4, 5 100% Recovery
9	- trace gravel (rounded, 20 mm) at 9.1 m	☒	S09	11	SPT Blows: 3, 5, 6 100% Recovery
10					
11	- fissuring at 10.7 m - trace silt, sand, and gravel below 10.7m	☒	S10	10	SPT Blows: 4, 7, 3 100% Recovery
12	SILT (TILL) - sandy, some clay, trace gravel - tan, wet, compact	☒	S11	17	SPT Blows: 3, 6, 11 100% Recovery
13		■	G12		
			S13	50/ 51mm	SPT Blows: 50/51mm, No Recovery
14	END OF TEST HOLE AT 13.8 m IN SILT (TILL)				





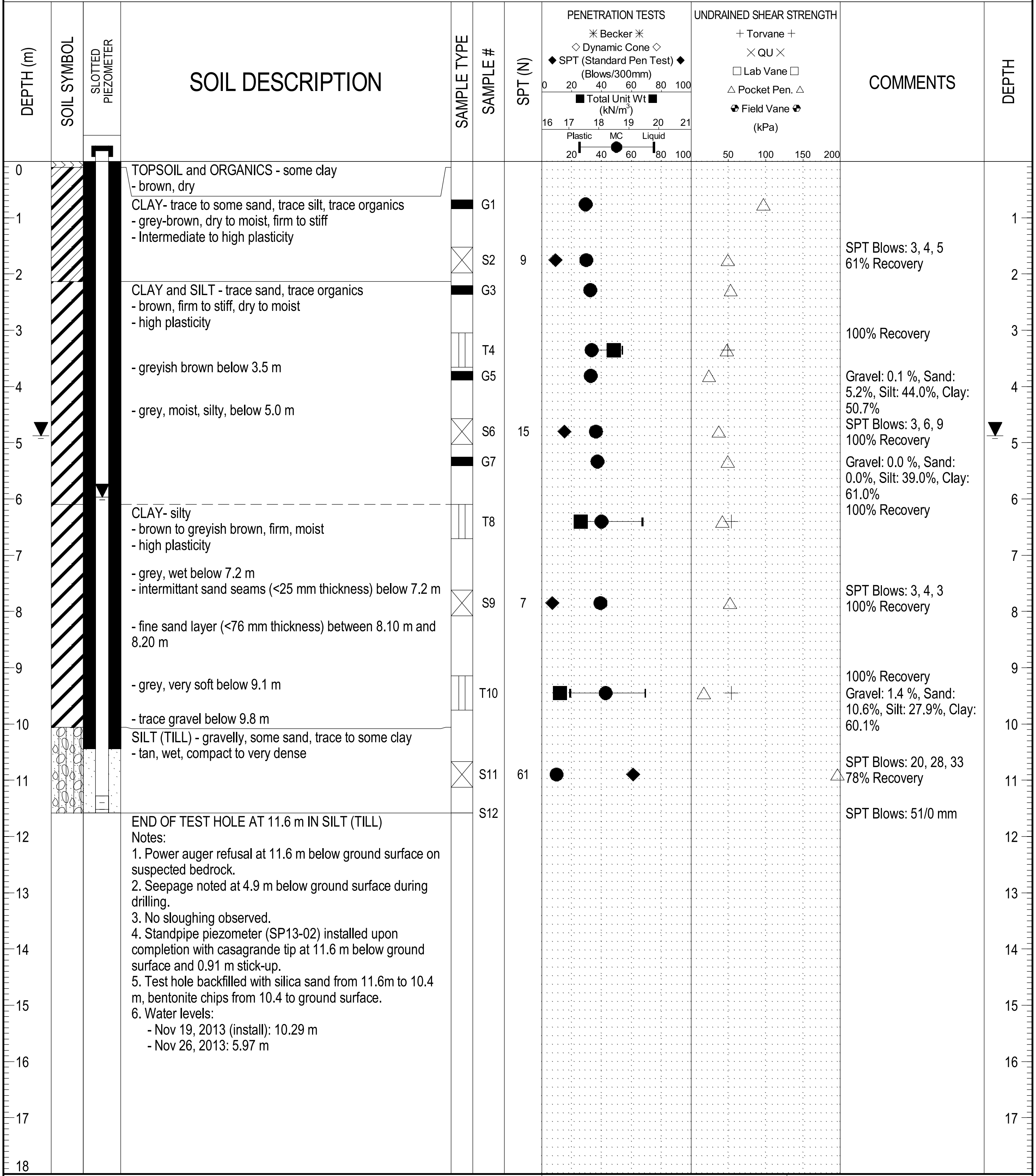
Notes:
1. Power auger refusal at 13.8 m below ground surface.
2. Seepage noted at 6.7 m below ground surface during drilling.
3. Sloughing not observed.
4. Standpipe piezometer (SP13-01) installed upon completion with casagrande tip at 13.7 m below ground surface and 0.9 m stick-up.
5. Test hole backfilled with silica sand from 13.7m to 11.3 m, bentonite chips from 11.3 to 6.1 m, auger cuttings from 6.1 to 1.2 m, and bentonite chips from 1.2 m to surface.
6. Water levels:
- Nov 8, 2013 (install): 12.95 m
- Nov 19, 2013: 5.70 m
- Nov 26, 2013: 6.02 m

AECOM	LOGGED BY: Aaron Kaluzniak	COMPLETION DEPTH: 13.76 m
	REVIEWED BY: Alex Hill	COMPLETION DATE: 11/8/13
	PROJECT ENGINEER: Marvin McDonald	

PROJECT: FGSV Interceptor Siphon	CLIENT: City of Winnipeg	TESTHOLE NO: **TH13-02**
LOCATION: Lower Bank of Red River, UTM: 14 U, N 5520490, E 0633691		PROJECT NO.: 60274906
CONTRACTOR: Paddock Drilling	METHOD: Truck Mounted Acker SS-3	ELEVATION (m):

SAMPLE TYPE: ■ GRAB | SHELBY TUBE | ☒ SPLIT SPOON | BULK | NO RECOVERY | CORE

BACKFILL TYPE: ■ BENTONITE | GRAVEL | SLOUGH | GROUT | CUTTINGS | SAND



SOIL DESCRIPTION

0–0.6 m: TOPSOIL and ORGANICS - some clay
- brown, dry

0.6–2.1 m: CLAY- trace to some sand, trace silt, trace organics
- grey-brown, dry to moist, firm to stiff
- Intermediate to high plasticity

2.1–6.1 m: CLAY and SILT - trace sand, trace organics
- brown, firm to stiff, dry to moist
- high plasticity
- greyish brown below 3.5 m
- grey, moist, silty, below 5.0 m

6.1–10.1 m: CLAY- silty
- brown to greyish brown, firm, moist
- high plasticity
- grey, wet below 7.2 m
- intermittant sand seams (<25 mm thickness) below 7.2 m
- fine sand layer (<76 mm thickness) between 8.10 m and 8.20 m
- grey, very soft below 9.1 m
- trace gravel below 9.8 m

10.1–11.6 m: SILT (TILL) - gravelly, some sand, trace to some clay
- tan, wet, compact to very dense

SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS
GRAB	G1		
SPLIT SPOON	S2	9	SPT Blows: 3, 4, 5 61% Recovery
GRAB	G3		
SHELBY TUBE	T4		100% Recovery
GRAB	G5		Gravel: 0.1 %, Sand: 5.2%, Silt: 44.0%, Clay: 50.7%
SPLIT SPOON	S6	15	SPT Blows: 3, 6, 9 100% Recovery
GRAB	G7		Gravel: 0.0 %, Sand: 0.0%, Silt: 39.0%, Clay: 61.0%
SHELBY TUBE	T8		100% Recovery
SPLIT SPOON	S9	7	SPT Blows: 3, 4, 3 100% Recovery
SHELBY TUBE	T10		100% Recovery Gravel: 1.4 %, Sand: 10.6%, Silt: 27.9%, Clay: 60.1%
SPLIT SPOON	S11	61	SPT Blows: 20, 28, 33 78% Recovery
	S12		SPT Blows: 51/0 mm

END OF TEST HOLE AT 11.6 m IN SILT (TILL)

Notes:

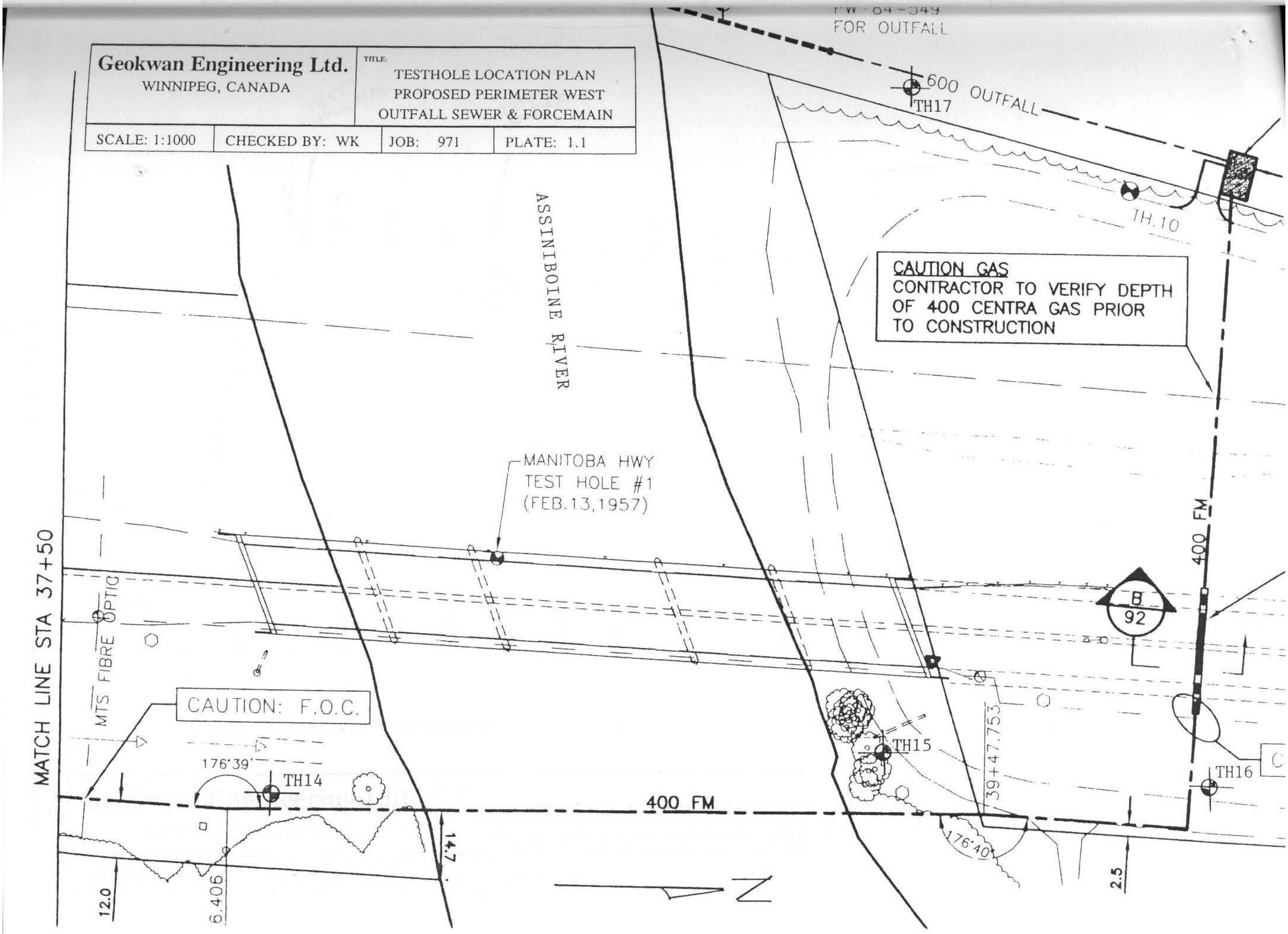
1. Power auger refusal at 11.6 m below ground surface on suspected bedrock.
2. Seepage noted at 4.9 m below ground surface during drilling.
3. No sloughing observed.
4. Standpipe piezometer (SP13-02) installed upon completion with casagrande tip at 11.6 m below ground surface and 0.91 m stick-up.
5. Test hole backfilled with silica sand from 11.6m to 10.4 m, bentonite chips from 10.4 to ground surface.
6. Water levels:
   - Nov 19, 2013 (install): 10.29 m
   - Nov 26, 2013: 5.97 m

AECOM	LOGGED BY: Sam Oshati	COMPLETION DEPTH: 11.58 m
	REVIEWED BY: Alex Hill	COMPLETION DATE: 11/19/13
	PROJECT ENGINEER: Marvin McDonald	Page 1 of 1

Geokwan Engineering Ltd.
WINNIPEG, CANADA

TITLE:
TESTHOLE LOCATION PLAN
PROPOSED PERIMETER WEST
OUTFALL SEWER & FORCEMAIN

SCALE: 1:1000	CHECKED BY: WK	JOB: 971	PLATE: 1.1



CAUTION GAS
CONTRACTOR TO VERIFY DEPTH
OF 400 CENTRA GAS PRIOR
TO CONSTRUCTION

MANITOBA HWY
TEST HOLE #1
(FEB.13,1957)

CAUTION: F.O.C.

TH14 (Elev. 234.565m)

0 - 4.88m CLAY
- firm, brown
- crumbly, desiccated, some organics to 0.3m
- trace to some gypsum & silt inclusions below 0.3m
- stiff below 1.5m, firm below 3.8m
- trace gravel below 2.3m, highly plastic

4.88 - 7.62m GLACIAL TILL
- soft to very soft
- clayey, wet to saturated, slight seepage
- medium dense below 6.4m
- silty, sandy, gravelly
- trace of suspected cobble/boulder

End of testhole at 7.62m from grade.

Note: Groundwater table at 7.54m from grade upon completion of drilling.

Depth (m)	Soil Water Content (%)	Penetrometer Reading (kPa)
0.76	36.6	75
1.52	39.1	125
2.28	41.0	130
3.05	40.5	130
3.81	-	75
4.57	41.1	75
4.88	-	0
5.33	20.8	0
6.10	16.5	30
6.86	9.8	-
7.62	10.8	-

TH15 (Elev. 233.350m)

0 - 3.00m FILL
- clay, stiff, desiccated
- sandy 2.7 - 3m
- crumbly, trace gravel to 1.5m
- trace organics to 2.7m
- some gravel from 1.5m to 2.7m
- soft and wet below 2.7m
- trace gypsum & silt inclusions

3.00 - 5.18m **CLAY & SILT**
- soft, sandy
- saturated, heavy seepage & very soft below 3.7m
- fill-like structure & trace rootlets to 3.9m
- grey at 4.5m

5.18 - 7.93m **GLACIAL TILL**
- medium dense
- silty, sandy, gravelly
- trace of suspected boulders below 5.5m

End of testhole at 7.93m from grade.

Note: Groundwater table at 4.04m and testhole caved to 4.11m from grade upon completion of drilling.

Depth (ft)	Soil Water Content (%)	Penetrometer Reading (kPa)
0.76	11.9	150
1.52	15.2	200
2.28	28.2	300
3.05	34.0	50
3.81	15.5	0
4.57	27.2	0
5.33	9.7	175
6.10	7.5	-
6.71	9.1	-
7.93	10.0	-

TH16 (Elev. 233.865m)

0 - 0.91m **FILL**
- clay, gravel & organics

0.91 - 4.30m **CLAY**
- very stiff to stiff
- black, brown & silty below 1.5m
- trace gypsum & silt inclusions
- soft, sandy & trace gravel below 3.1m
- wet to saturated at 4.2m

4.30 - 6.00m **SAND & GRAVEL**
- heavy seepage
- some silt & clay

6.00	-	7.62m	GLACIAL TILL - medium dense - silty, sandy, gravelly - trace boulders below 7m

End of testhole at 7.62m from grade.

Note: Groundwater table at 3.66m and testhole caved to 5.8m from grade upon completion of drilling.

Depth (m)	Soil Water Content (%)	Penetrometer Reading (kPa)
0.76	32.1	275
1.22	-	215
1.52	23.8	175
2.28	32.2	260
3.05	30.1	250
3.20	-	50
3.81	-	-
4.57	22.8	50
5.33	16.7	0
6.10	8.3	-
6.86	10.4	-

TH17 (Elev. 233.383m)

0	-	0.61m	TOPSOIL - soft, brown, organics
0.61	-	3.20m	CLAY - very stiff, dark brown - stiff, brown, silty, trace gypsum & silt inclusions below 1.1m
3.20	-	3.35m	SAND - fine to medium grained, wet to saturated, moderate seepage
3.35	-	3.51m	CLAY - soft, silty, brown, trace gypsum & silt inclusions
3.51	-	4.11m	SAND & GRAVEL - medium to coarse grained, heavy seepage
4.11	-	5.33m	CLAY - soft, silty, grey, trace gypsum & silt inclusions

5.33 - 7.62m GLACIAL TILL
- medium dense
- silty, sandy, gravelly
- trace of suspected cobble/boulder

End of testhole at 7.62m from grade.

Note: Groundwater table at 3.66m and testhole caved to 4.42m from grade upon completion of drilling.

Depth (ft)	Soil Water Content (%)	Penetrometer Reading (tsf)
0.76	26.8	325
1.52	27.7	175
2.28	29.3	175
3.05	29.0	150
3.43	-	100
4.57	59.6	0
5.33	10.0	125
6.10	9.5	125
6.86	8.1	-

TH18 (Elev. 234.606m)

0 - 4.57m CLAY
- very stiff, brown
- stiff at 2.28m, soft below 3m
- crumbly, desiccated to 1.8m
- trace of some organics to 1.8m
- silty, some gypsum & silt inclusions
- sandy to 3m
- frequent sand seams, moderate to heavy seepage below 3m

4.57 - 5.49m SAND & GRAVEL
- medium to coarse grained, saturated, heavy seepage

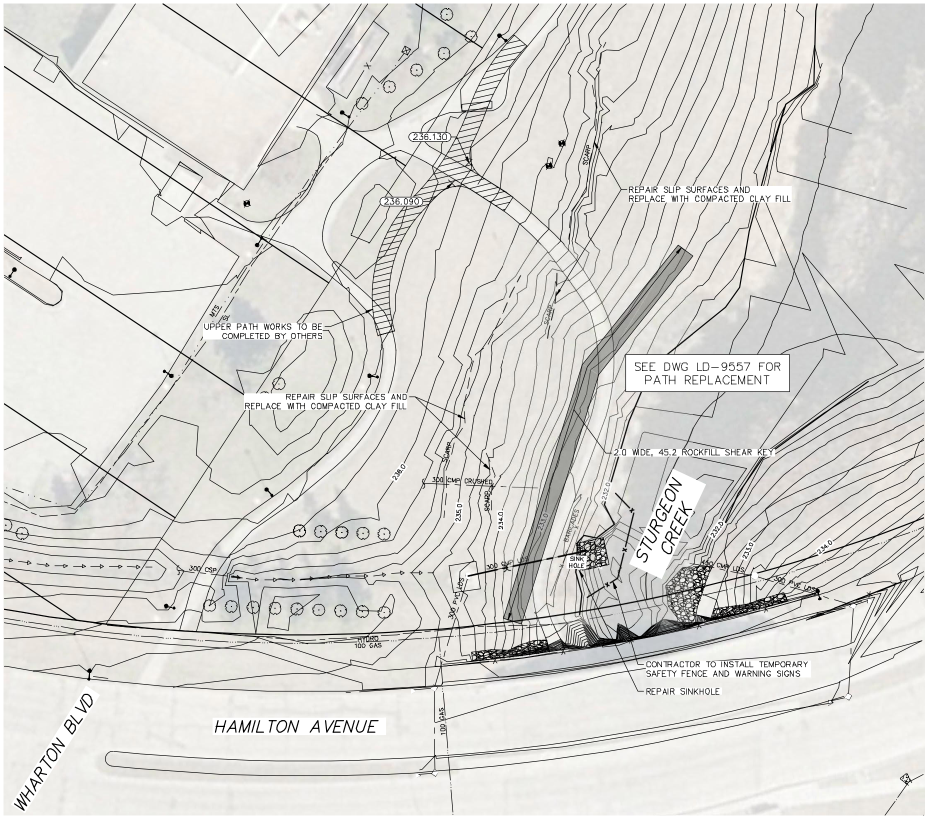
5.49 - 6.40m CLAY
- firm, soft below 6.2m
- grey, trace gypsum & silt inclusions

6.40 - 7.62m GLACIAL TILL
- soft, clayey, saturated, moderate seepage to 6.8m
- medium dense to dense below 6.8m
- silty, sandy, gravelly
- trace of suspected cobble/boulder

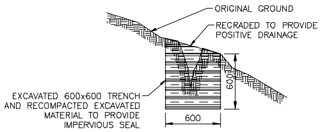
End of testhole at 7.62m from grade.

Note: Groundwater table at 4.42m and testhole caved to 4.72m from grade upon completion of drilling.

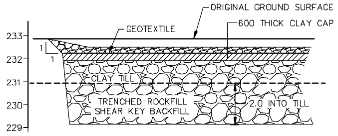
Depth (ft)	Soil Water Content (%)	Penetrometer Reading (tsf)
0.76	13.2	400
1.52	11.4	300
2.28	25.9	125
3.05	24.3	125
4.57	29.0	0
6.10	51.6	75
6.40	21.1	0
6.86	9.2	-
6.62	7.3	-



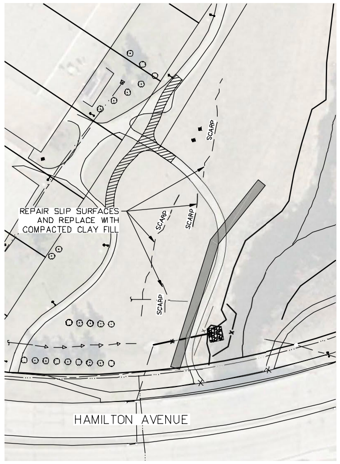
PLAN
SCALE 1:250



SLIP SURFACE REPAIR DETAIL
SCALE: N.T.S.



TRENCHED ROCKFILL SHEAR KEY
SCALE: NTS



SLIP SURFACE REPAIR PLAN
SCALE: NTS

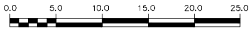
NOTES:

1. ALL EXCAVATED MATERIAL WITHIN 76.0 OF THE RSRL MUST BE REMOVED FROM SITE (NO STOCKPILING).
2. CONSTRUCT CLAY COFFERDAM AT CREEK AS REQUIRED (SEE DWG LD-9558).
3. CONTRACTOR TO PROVIDE AND INSTALL PROPER WARNING SIGNS ALONG PEDESTRIAN PATHWAY CONCERNING WORK SITE.

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FOR INDEX PAGE
SEE DWG LD-9554

METRIC
WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES



WARNING

IF POWER EQUIPMENT OR EXPLOSIVES ARE TO BE USED FOR EXCAVATION ON THIS PROJECT THE CONTRACTOR MUST:

1) NOTIFY THE GAS COMPANY OF THE PROPOSED LOCATION OF EXCAVATION.

2) TAKE PRECAUTION TO AVOID DAMAGE TO GAS COMPANY INSTALLATIONS.

SEE PROVINCIAL REGULATION 210/72 FOR DETAILS

LOCATION APPROVED
UNDERGROUND STRUCTURES

SUPV. U/G STRUCTURES COMMITTEE
DATE

NOTE:
LOCATION OF UNDERGROUND STRUCTURES AS SHOWN ARE BASED ON THE BEST INFORMATION AVAILABLE BUT NO GUARANTEE IS GIVEN THAT ALL EXISTING UTILITIES ARE SHOWN OR THAT THE GIVEN LOCATIONS ARE EXACT. CONFIRMATION OF EXISTENCE AND EXACT LOCATION OF ALL SERVICES MUST BE OBTAINED FROM THE INDIVIDUAL UTILITIES BEFORE PROCEEDING WITH CONSTRUCTION.

B.M.
ELEV.

NO.	REVISIONS	DATE	BY
A	ISSUED FOR REVIEW	2019 10 10	RBD

KGS GROUP
CONSULTING ENGINEERS

DESIGNED BY NGV
CHECKED BY
DRAWN BY GEL
APPROVED BY
SCALE:
HORIZONTAL 1:250
VERTICAL 1:50
RELEASED FOR CONSTRUCTION
DATE 2019 10 10
DATE
PLOT DATE: 2019 10 10

ENGINEER'S SEAL

CONSULTANT DRAWING NUMBER
LD-9556

BID OPPORTUNITY: 1042-2018
CONTRACT NUMBER:

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT
ENGINEERING DIVISION

HAMILTON AVENUE BRIDGE OUTFALLS
RENEWAL AND REHABILITATION
HAMILTON AVENUE BRIDGE OUTFALLS
GEOTECHNICAL WORKS

SHEET 4 OF 7
CITY DRAWING NUMBER
LD-9556

FILE PATH: P:\Projects\2019\19-0107-008\Dwg\Mun\
FILE NAME: 19-0107-008_PP.dwg

KGS GROUP

SUMMARY LOG

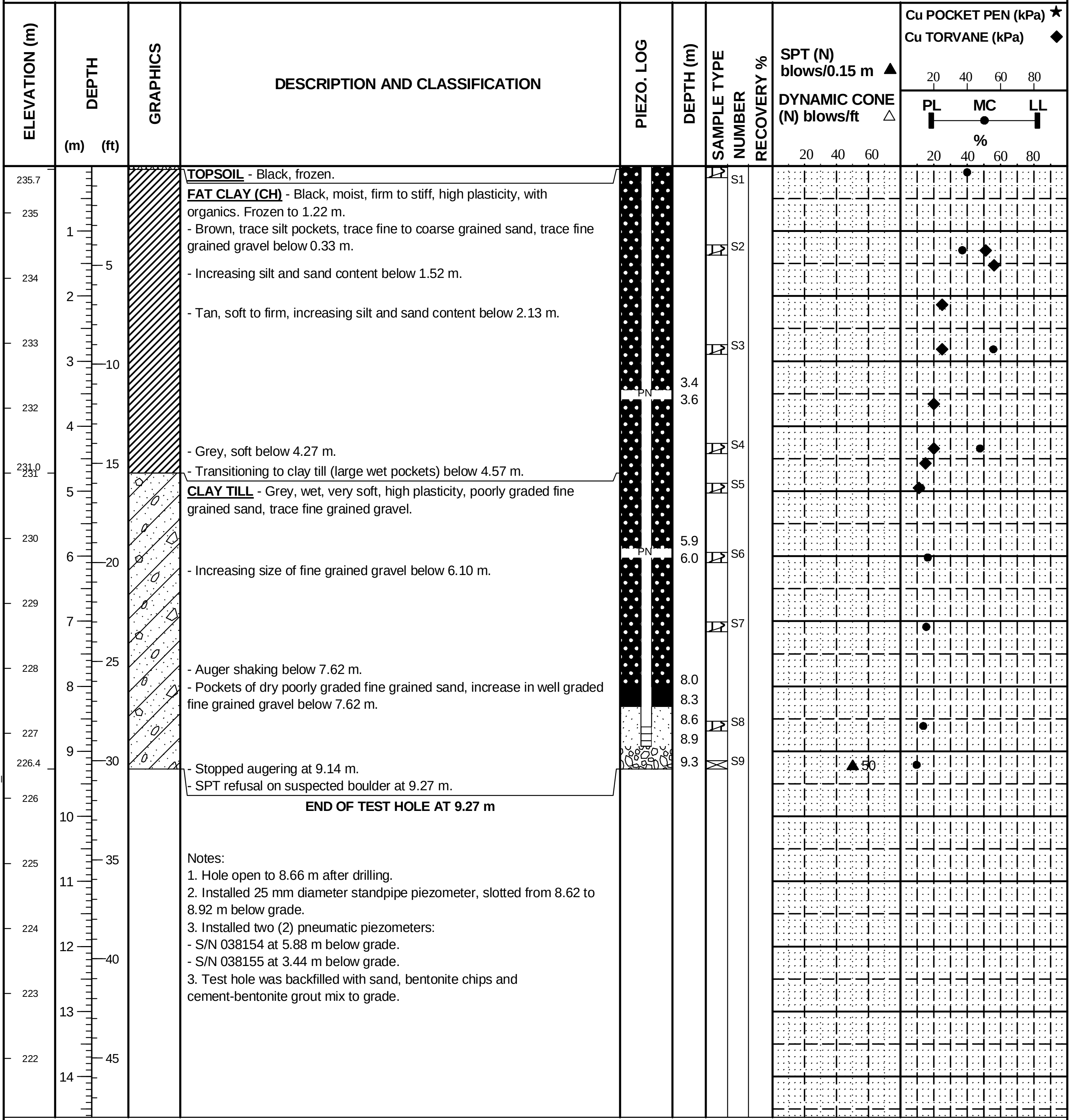
REFERENCE NO.

HOLE NO.
TH19-03

SHEET 1 of 1

CLIENT MANITOBA HOUSING & RENEWAL CORP.
PROJECT Bruce Oake Recovery Center
SITE 255 Hamilton Avenue, Winnipeg, Manitoba
LOCATION Mid Bank of Sturgeon Creek
DRILLING METHOD 125 mm ø Solid Stem Auger, Acker MP5-T

JOB NO. 18-1441-006
GROUND ELEV. 235.72
TOP OF CASING ELEV. 236.86
WATER ELEV.
DATE DRILLED 4/5/2019
UTM (m) N 5,528,218
E 622,855



TOPSOIL - Black, frozen.

FAT CLAY (CH) - Black, moist, firm to stiff, high plasticity, with organics. Frozen to 1.22 m.
- Brown, trace silt pockets, trace fine to coarse grained sand, trace fine grained gravel below 0.33 m.

- Increasing silt and sand content below 1.52 m.

- Tan, soft to firm, increasing silt and sand content below 2.13 m.

- Grey, soft below 4.27 m.
- Transitioning to clay till (large wet pockets) below 4.57 m.

CLAY TILL - Grey, wet, very soft, high plasticity, poorly graded fine grained sand, trace fine grained gravel.

- Increasing size of fine grained gravel below 6.10 m.

- Auger shaking below 7.62 m.
- Pockets of dry poorly graded fine grained sand, increase in well graded fine grained gravel below 7.62 m.

- Stopped augering at 9.14 m.
- SPT refusal on suspected boulder at 9.27 m.

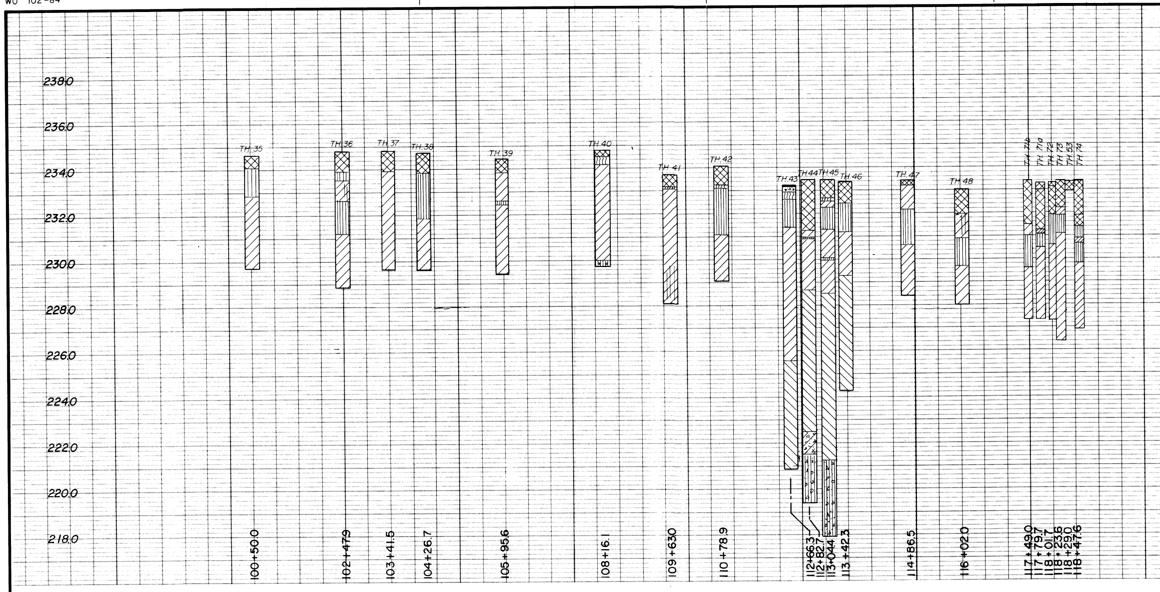
END OF TEST HOLE AT 9.27 m

Notes:
1. Hole open to 8.66 m after drilling.
2. Installed 25 mm diameter standpipe piezometer, slotted from 8.62 to 8.92 m below grade.
3. Installed two (2) pneumatic piezometers:
- S/N 038154 at 5.88 m below grade.
- S/N 038155 at 3.44 m below grade.
3. Test hole was backfilled with sand, bentonite chips and cement-bentonite grout mix to grade.

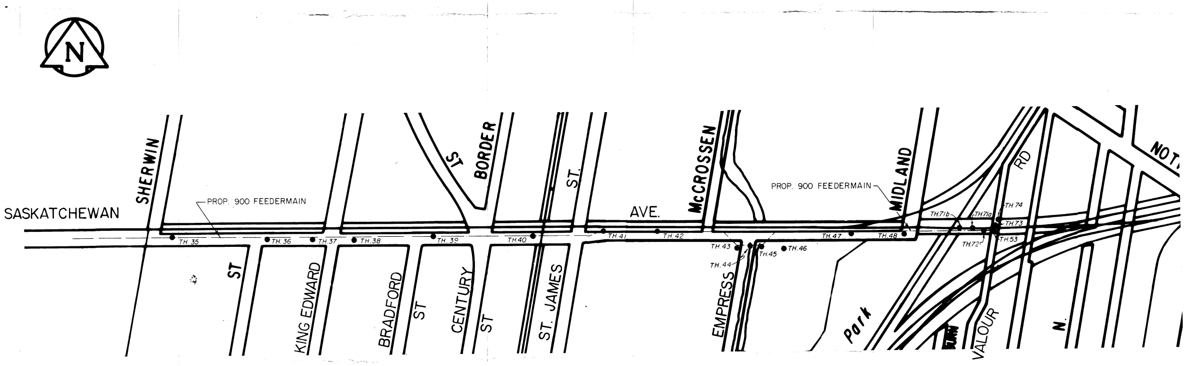
SAMPLE TYPE ⧉ Auger Grab ☒ Split Spoon

CONTRACTOR	INSPECTOR	APPROVED	DATE
Maple Leaf Enterprises	L.CHALMERS	D. ANDERSON	4/9/19

WO 102-84



TEST HOLE NO.	ELEVATION
35	234.66
36	234.812
37	234.82
38	234.686
39	234.42
40	234.79
41	233.70
42	234.04
43	233.22
44	233.44
45	233.43
46	233.35
47	233.41
48	233.04
71b	233.34
71a	233.30
72	233.33
73	233.40
53	233.34
74	233.40



LEGEND

▥	TOPSOIL
▩	FILL
▨	CLAY
▥	SILT
▨	TILL

B.M. ELEV.			
NO	REVISIONS	DATE	BY

WORKS & OPERATIONS DIVISION
WATERWORKS, WASTE & DISPOSAL DEPARTMENT

DESIGNED BY	CHECKED BY TH
DRAWN BY RN	APPROVED BY
HOR. SCALE 1:5000 VERTICAL 1:100	RELEASED FOR CONSTRUCTION
DATE JULY, 1987	DATE

ENGINEER'S SEAL
R. J. WITTEBOLLE

CONSULTANT DRAWING NO.

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION

WEST END FEEDERMAIN

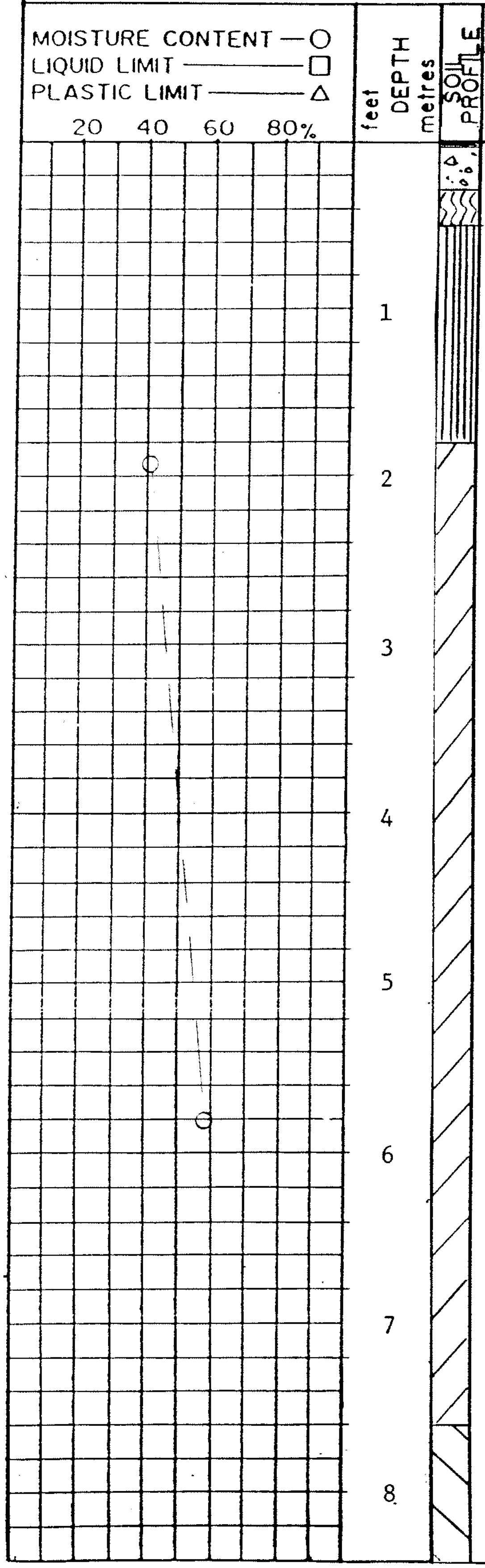
CITY DRAWING NUMBER
SHEET OF
03

uma **UMA Engineering Ltd.**
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 43

SURFACE ELEVATION: 233.22m
CO-ORDINATES: ______



SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH □psf □kPa	MISC TESTS AND REMARKS
50 MM ASPHALT				
GRAVEL (fill) - frozen				
CLAY - black (topsoil) - organic				
SILT - light brown - wet - soft				
	1B			γ_d= 12.40 KN/m^3 γ_w=17.46 KN/m^3 L_v=78.0 kPa
CLAY - brown - weathered in upper portion - some silt layering in upper portion - plastic - firm				
	2G			
	3B			γ_d=9.98 KN/m^3 γ_w=15.75 KN/m^3 L_v= 44.0 kPa

uma UMA Engineering Ltd.
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 43
Contin.

MOISTURE CONTENT — ○
LIQUID LIMIT — □
PLASTIC LIMIT — △
20 40 60 80%

SURFACE ELEVATION: 233.22m
CO-ORDINATES:

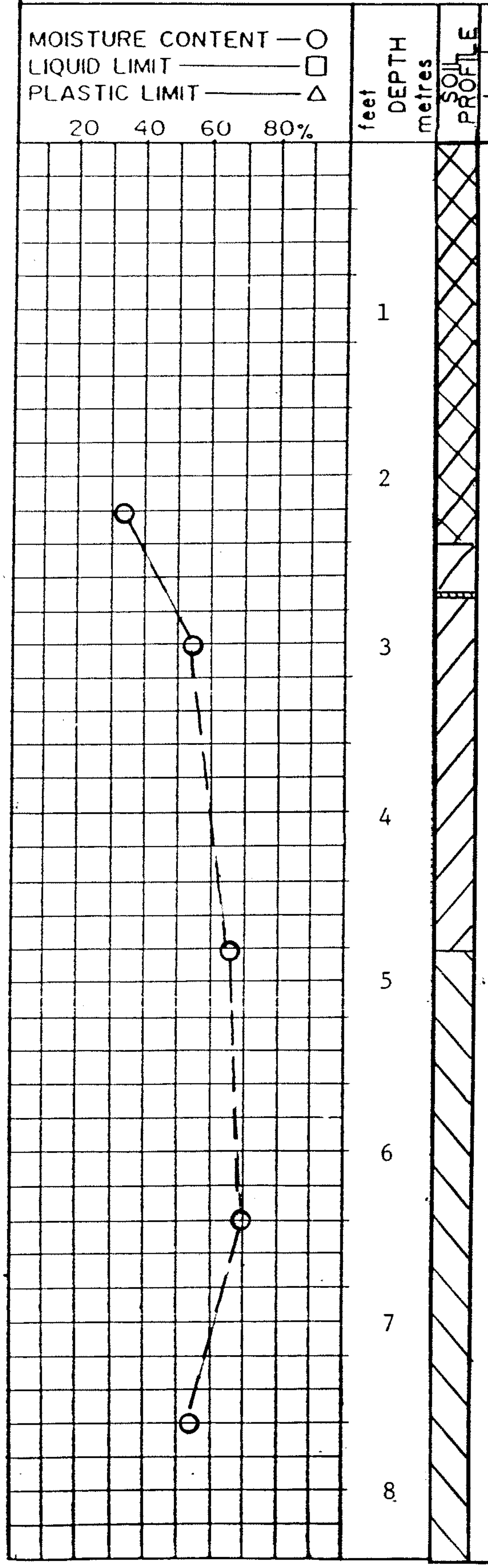
feet DEPTH metres	SOIL PROFILE	SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
	▨	CLAY - grey - till inclusions - firm to soft				
9	▨		4B			γ_d=11.48 KN/m^3 γ_w=17.00 KN/m^3 L_v=38 kPa
10	▨					
11	▨		5G			
12	▨					
		End of hole at 12.2 m.				
13						
14		NOTES: - no seepage during drilling.				
15						
16						

UMA Engineering Ltd.
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 44

SURFACE ELEVATION: 233.44m
CO-ORDINATES: ____________



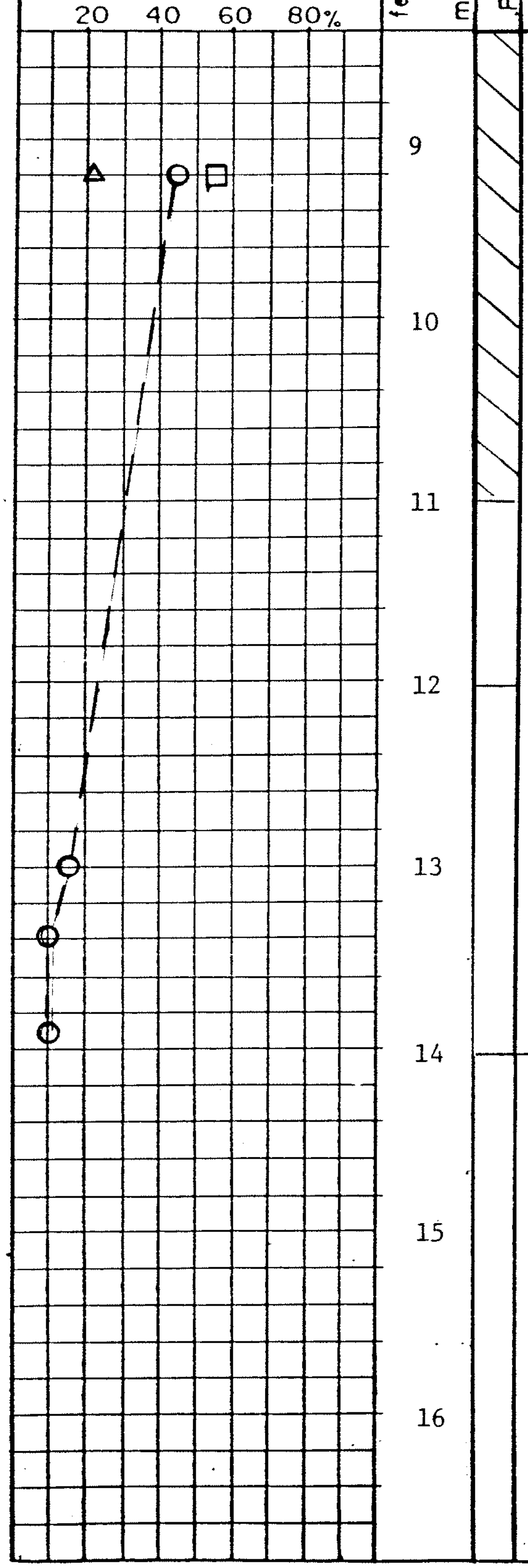
SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
FILL - clay - topsoil - silt - stiff to firm				
	1B			γ_d=13.49 KN/m^4 γ_w=18.06 KN/m^3 L_v=48.2 kPa
CLAY - brown - 75 mm silt layer at 2.7 m - stiff to firm with depth	2B			γ_d=10.51 KN/m^3 γ_w=16.31 KN/m^3 L_v= 68.9 kPa
	3B			γ_d=9.34 KN/m^3 γ_w=15.58 KN/m^3 L_v=47.9 kPa
CLAY - grey - trace of silt pockets - firm to stiff with depth	4B			
- till inclusions at 7.5 m	5B			γ_d=11.05 KN/m^3 γ_w=16.86 KN /m^3 L_v= 32.2 kPa

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 44 Contin.

SURFACE ELEVATION: 233.44m
CO-ORDINATES:

MOISTURE CONTENT — ○
LIQUID LIMIT — □
PLASTIC LIMIT — △



SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH □psf □kPa	MISC TESTS AND REMARKS
	6B			L_V=26.3 kPa PI = 33.2%
	7B			
CLAY (till) - soft - grey				
	8B			
SILT (till) - brown - sandy - with gravel - dense to very dense with depth	9G 10G 11G			
Auger refusal at 14.0 m.				

NOTES:
- water level ± 5 m from bottom of borehole after 20 minutes.

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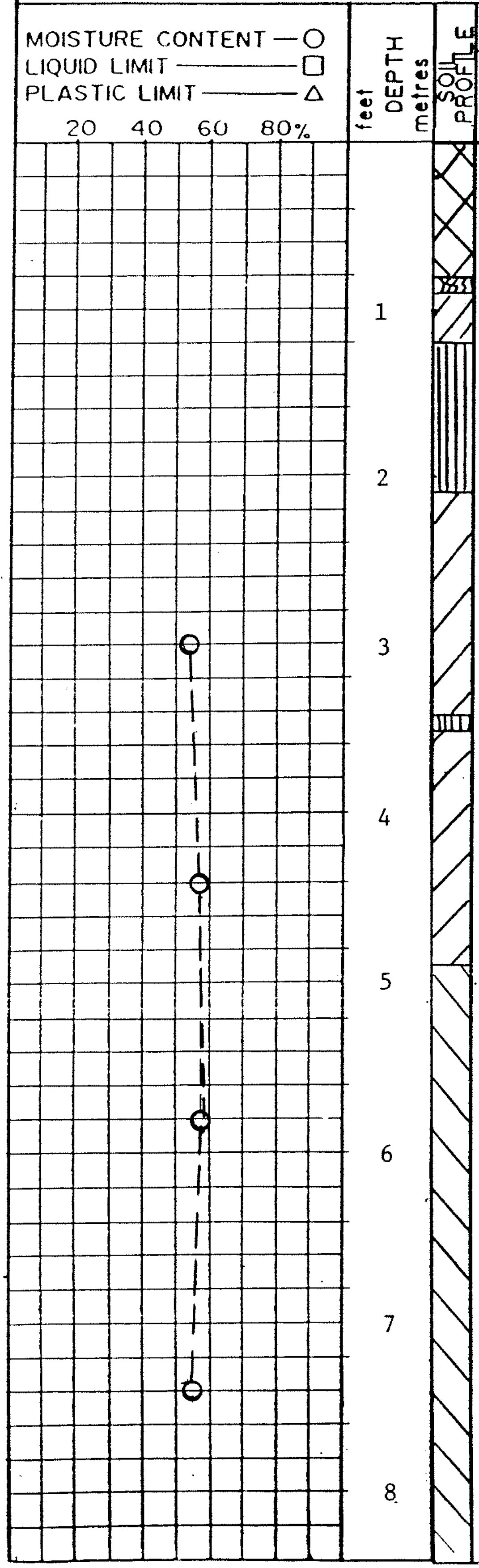
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 45

SURFACE ELEVATION: 233.43m
CO-ORDINATES:

MOISTURE CONTENT —○
LIQUID LIMIT —□
PLASTIC LIMIT —△



SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
FILL				
CLAY - black - organic				
SILT - light brown - wet - soft				
CLAY - brown - stratified - occasional thin silt layers - weathered - stiff	1B			PI = 54% γ_d=10.78 KN/m^3 γ_w=16.46 KN/m^3 L_v=65 kPa
SILT - oxidized				Apparent Slickenside @ 35° from horizontal
CLAY - brown - trace of small silt pockets - stiff to firm	2B			γ_d=10.35 KN/m^3 L_V=75.2 kPa
	3B			γ_d=10.55 KN/m^3 γ_w=16.55 KN/m^3 L_V=80.4 kPa
CLAY - grey - plastic - occasional till pockets - stiff to firm with depth				
	4B			γ_d=10.87 KN/m^3 γ_w=16.68 KN/m^3 L_V=27.9 kPa

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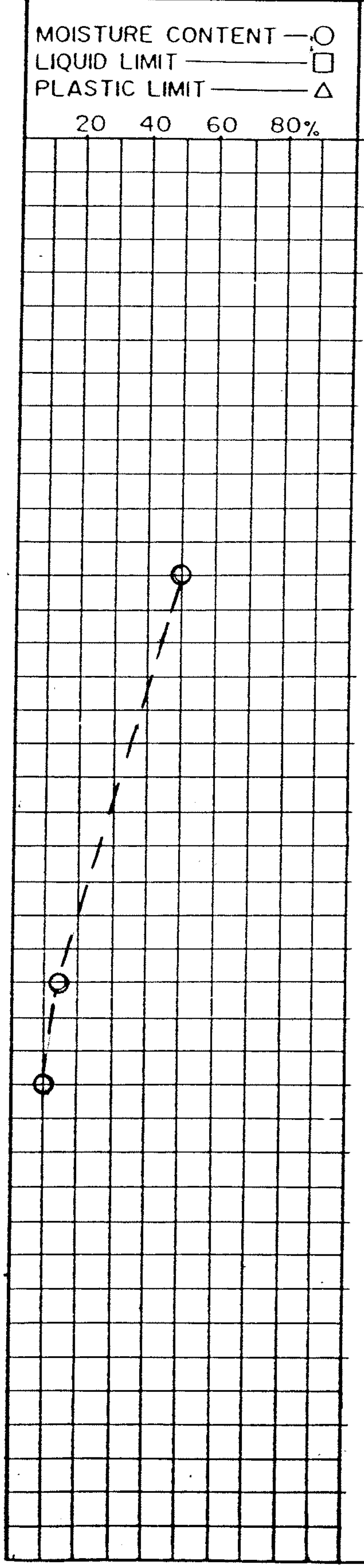
PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 45 Contin.

SURFACE ELEVATION: 233.43m
CO-ORDINATES:



DEPTH feet / metres	SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
9		5B			
10					
11		6B			γ_d=11.69 KN/m^3 γ_w=17.58 KN/m^3 L_v=40.1 kPa
12		7B			
13	TILL - silt and clay layers - sandy - some gravel - light brown - soft @ clay interface - dense to very dense with depth	8G			
14		9G			
15					
16	Auger refusal at 15.5 m.				

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1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 18, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 46

MOISTURE CONTENT — ○
LIQUID LIMIT — □
PLASTIC LIMIT — △
20 40 60 80%

SURFACE ELEVATION: 233.35m
CO-ORDINATES:

DEPTH feet metres	SOIL PROFILE	SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
		FILL - gravel and slag - frozen				
1		SILT - light brown - wet - soft				
2						
		CLAY - brown - highly plastic - trace of silt - damp - stiff to firm with depth				
3			B1			
			B2			
4						
5						
		CLAY - grey - highly plastic - moist - firm to soft at 4.6 mm - occasional pebble and trace of silt at 6.0 m				
6						
			B3			
7						
		- very soft at 7.5 m				
8						

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1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 46 Contin.

MOISTURE CONTENT — ○
LIQUID LIMIT — □
PLASTIC LIMIT — △
20 40 60 80%

DEPTH feet metres

SOIL PROFILE

SURFACE ELEVATION: 233.35m
CO-ORDINATES:

SOIL DESCRIPTION

SAMPLE NO.

STANDARD PEN.(N)

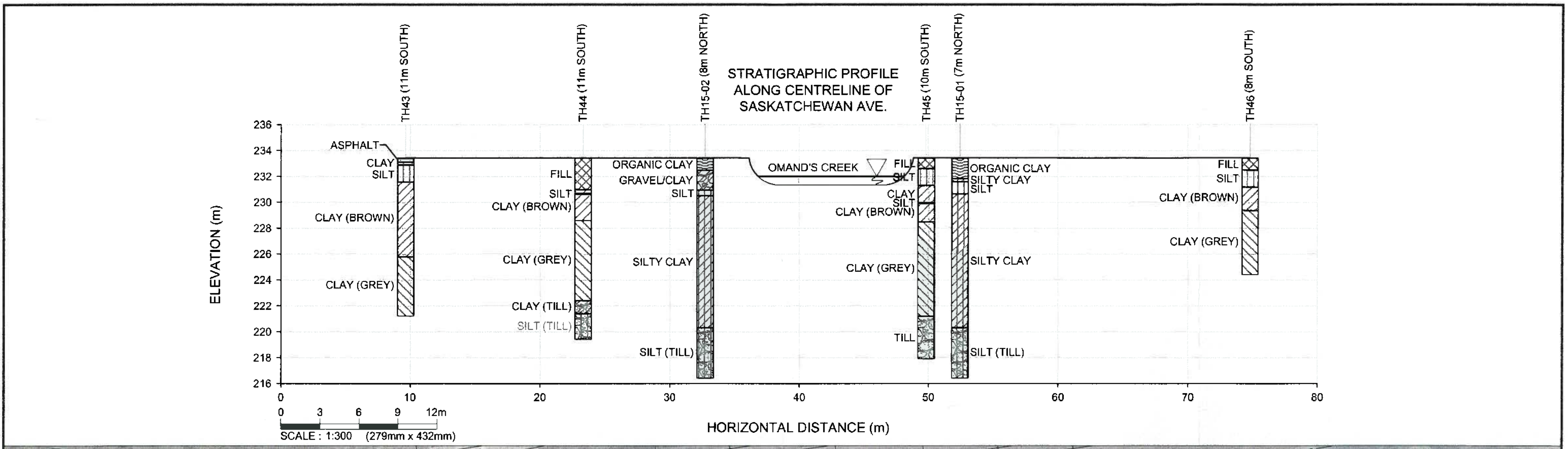
COMP. STRENGTH □psf □kPa

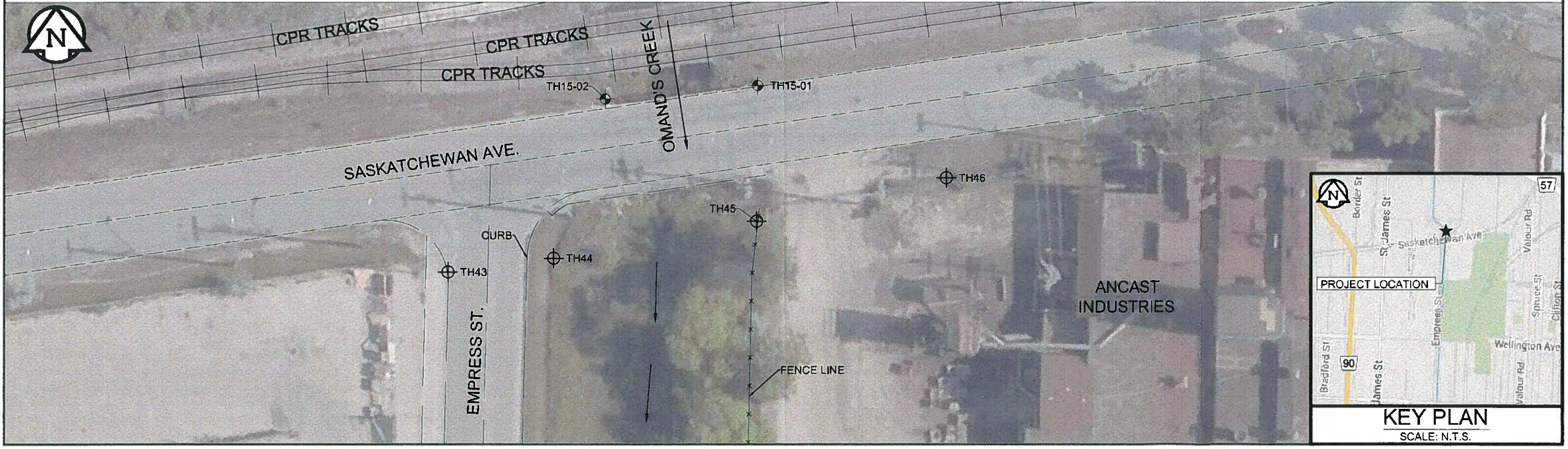
MISC TESTS AND REMARKS

DEPTH metres	SOIL DESCRIPTION	SAMPLE NO.
		G1
9	End of hole at 9.0 m.	
10	NOTES: - some sloughing from silt layer during drilling.	
11		
12		
13		
14		
15		
16		

TREK GEOTECHNICAL

0035 020 00
Morrison Hershfield
Saskatchewan Ave over Omand's Creek





0 5 10 15 20m
SCALE : 1:500 (279mm x 432mm)

LEGEND :

- ⊕ TEST HOLE (TREK, APRIL 7, 2015)

NOTES :

1. AERIAL IMAGE FROM BING MAP

Figure 01
Test Hole Location Plan
and Stratigraphic Profile

Tabloid (279mm x 432mm)

PLOT: 6/17/2015 2:12:13 PM

FILE NAME: FIG 001 2015-06-17 Site Plan 0_F_HA 0035 020 00.dwg

TREK GEOTECHNICAL

Sub-Surface Log

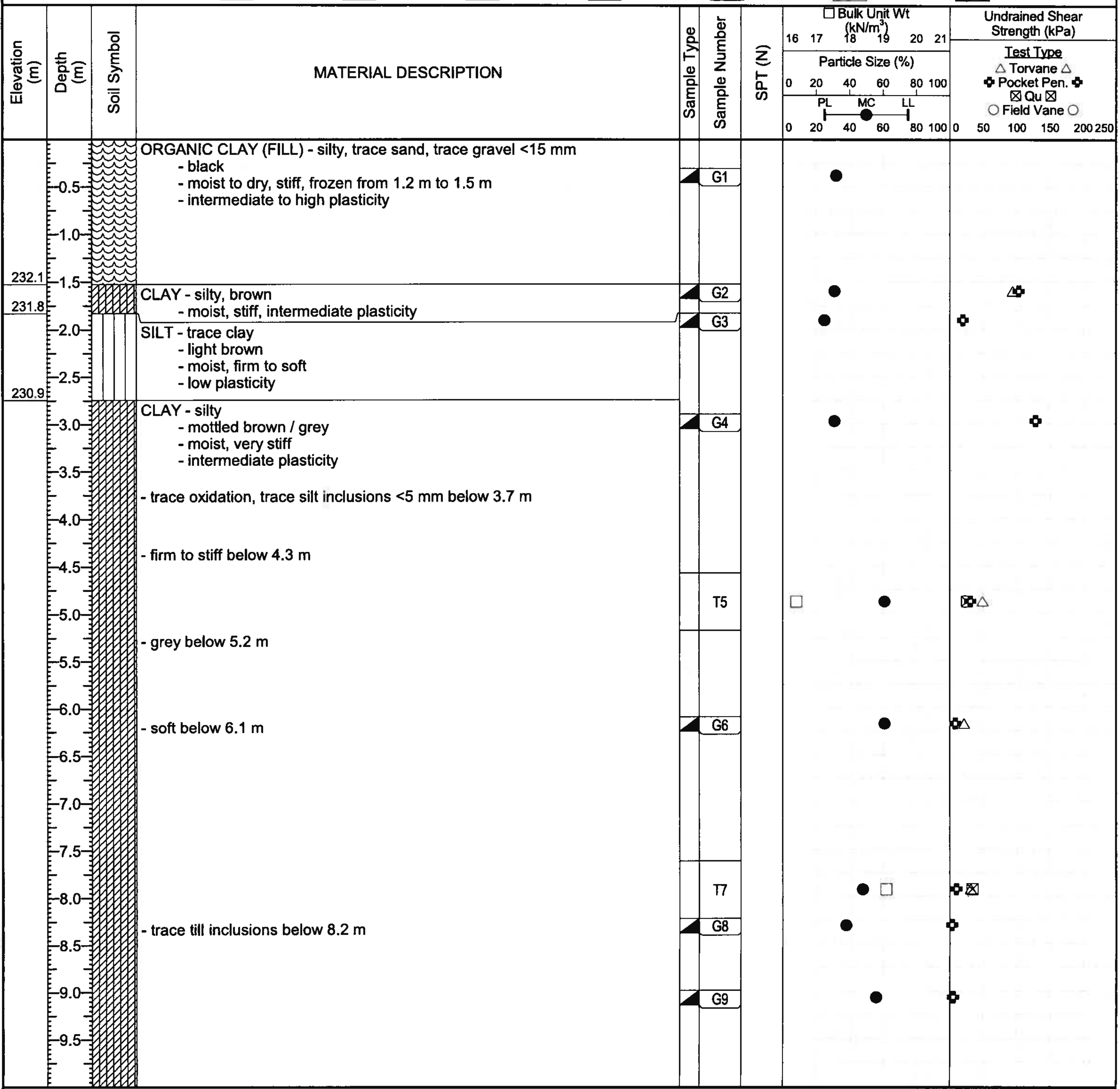
Test Hole TH15-01

1 of 2

Client:	Morrison Hershfield	**Project Number:**	0035 020 00
Project Name:	Saskatchewan over Omand's Creek	**Location:**	UTM N-5529845.75, E-629659.55
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	233.66 m Existing Ground
Method:	125 mm Solid Stem Auger, B37X Track Mount	**Date Drilled:**	7 April 2015

Sample Type: ◢ Grab (G) ▌ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ▯ Core (C)

Particle Size Legend: ▩ Fines ▨ Clay ▥ Silt ⛆ Sand Gravel Cobbles Boulders



ORGANIC CLAY (FILL) - silty, trace sand, trace gravel <15 mm
- black
- moist to dry, stiff, frozen from 1.2 m to 1.5 m
- intermediate to high plasticity

CLAY - silty, brown
- moist, stiff, intermediate plasticity

SILT - trace clay
- light brown
- moist, firm to soft
- low plasticity

CLAY - silty
- mottled brown / grey
- moist, very stiff
- intermediate plasticity

- trace oxidation, trace silt inclusions <5 mm below 3.7 m

- firm to stiff below 4.3 m

- grey below 5.2 m

- soft below 6.1 m

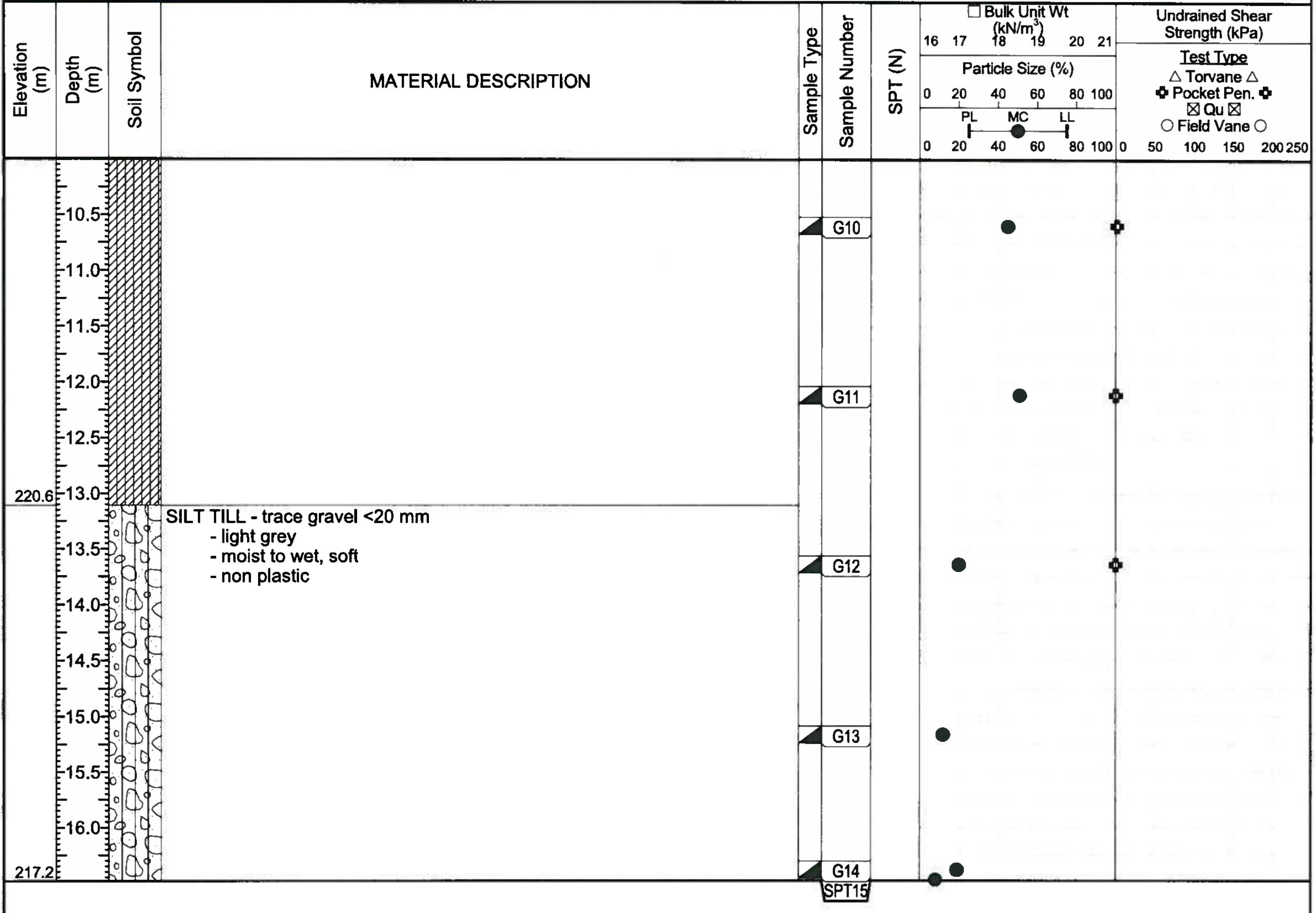
- trace till inclusions below 8.2 m

Logged By: Syl Precourt **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

Sub-Surface Log

Test Hole TH15-01

2 of 2



SILT TILL - trace gravel <20 mm
- light grey
- moist to wet, soft
- non plastic

END OF TEST HOLE AT 16.5 m IN SILT TILL

Notes:

1) Power auger refusal encountered at 16.5 m.

2) No seepage or sloughing observed.

3) Water at 6.7 m

4) Test hole was backfilled with auger cuttings 0.5 m bentonite at bottom of test hole and 0.5 m bentonite at top

5) Test hole was open to 11.6 m

Logged By: Syl Precourt **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

Sub-Surface Log

Test Hole TH15-02

1 of 2

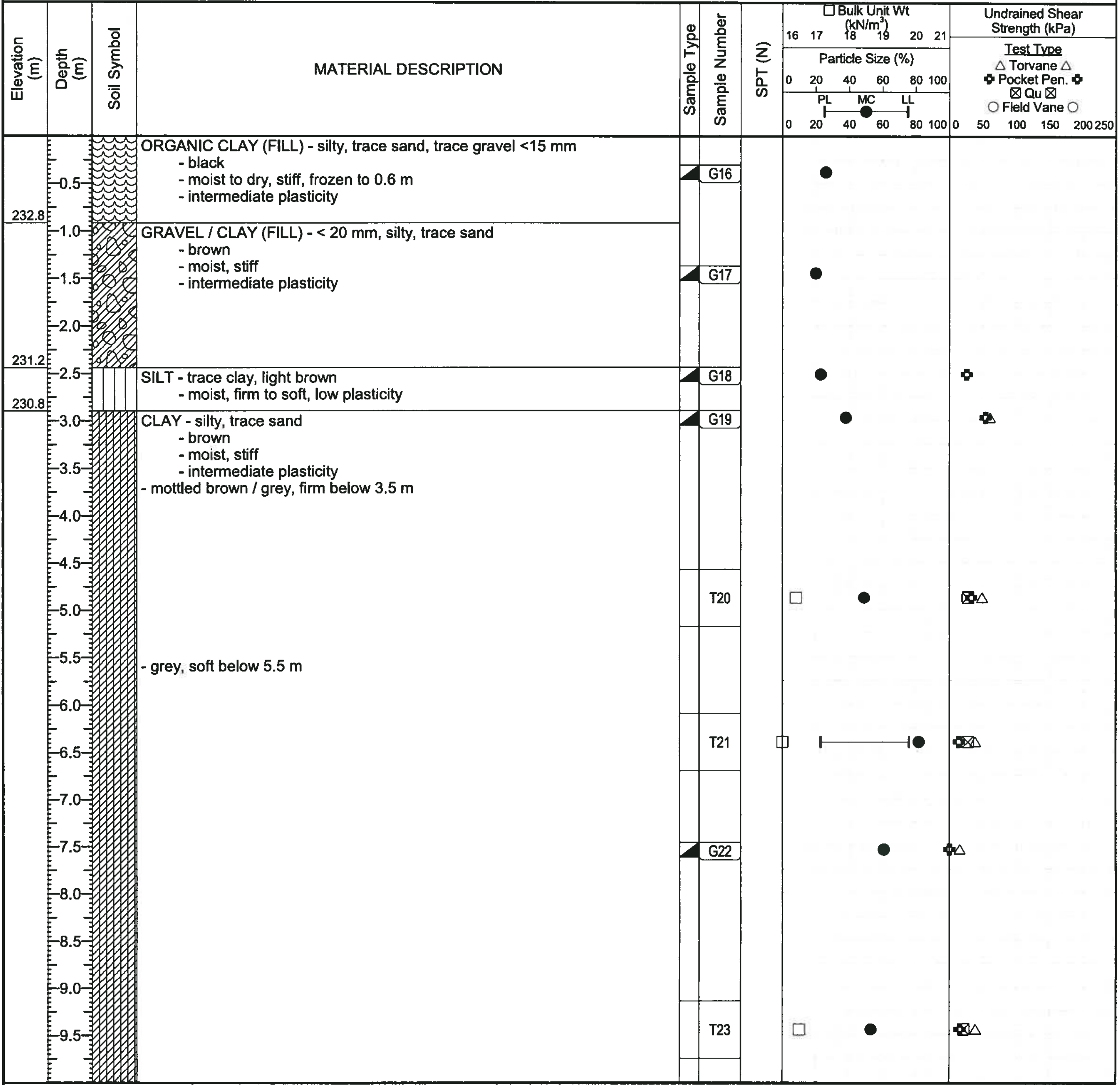
Client:	Morrison Hershfield	**Project Number:**	0035 020 00
Project Name:	Saskatchewan over Omand's Creek	**Location:**	UTM N-5529842.53, E-629636.11
Contractor:	Maple Leaf Drilling	**Ground Elevation:**	233.68 m Existing Ground
Method:	125 mm Solid Stem Auger, B37X Track Mount	**Date Drilled:**	7 April 2015

Sample Type: ◩ Grab (G) ▮ Shelby Tube (T) ☒ Split Spoon (SS) ⧗ Split Barrel (SB) ▯ Core (C)

Particle Size Legend: ▦ Fines ▨ Clay ▥ Silt ░ Sand Gravel Cobbles Boulders

Elevation (m)	Depth (m)	MATERIAL DESCRIPTION
	0.0	ORGANIC CLAY (FILL) - silty, trace sand, trace gravel <15 mm - black - moist to dry, stiff, frozen to 0.6 m - intermediate plasticity
232.8		GRAVEL / CLAY (FILL) - < 20 mm, silty, trace sand - brown - moist, stiff - intermediate plasticity
231.2		SILT - trace clay, light brown - moist, firm to soft, low plasticity
230.8		CLAY - silty, trace sand - brown - moist, stiff - intermediate plasticity - mottled brown / grey, firm below 3.5 m - grey, soft below 5.5 m

Sample Type	Sample Number
◩	G16
◩	G17
◩	G18
◩	G19
	T20
	T21
◩	G22
	T23



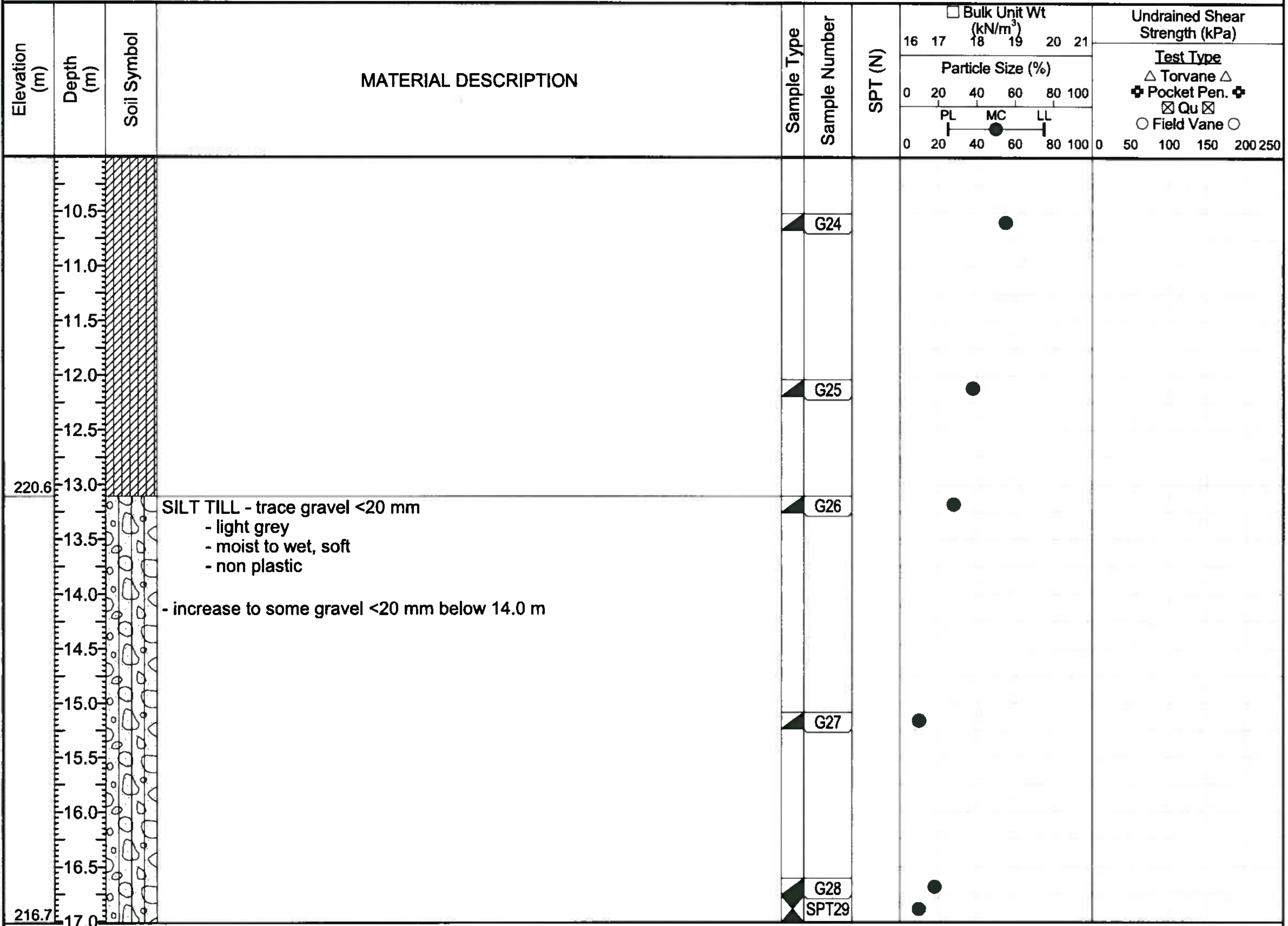
Logged By: Syl Precourt **Reviewed By:** Michael Van Helden **Project Engineer:** Michael Van Helden

TREK GEOTECHNICAL

Sub-Surface Log

Test Hole TH15-02

2 of 2



SILT TILL - trace gravel <20 mm
- light grey
- moist to wet, soft
- non plastic

- increase to some gravel <20 mm below 14.0 m

END OF TEST HOLE AT 16.7 m IN SILT TILL ON SUSPECTED BOULDER

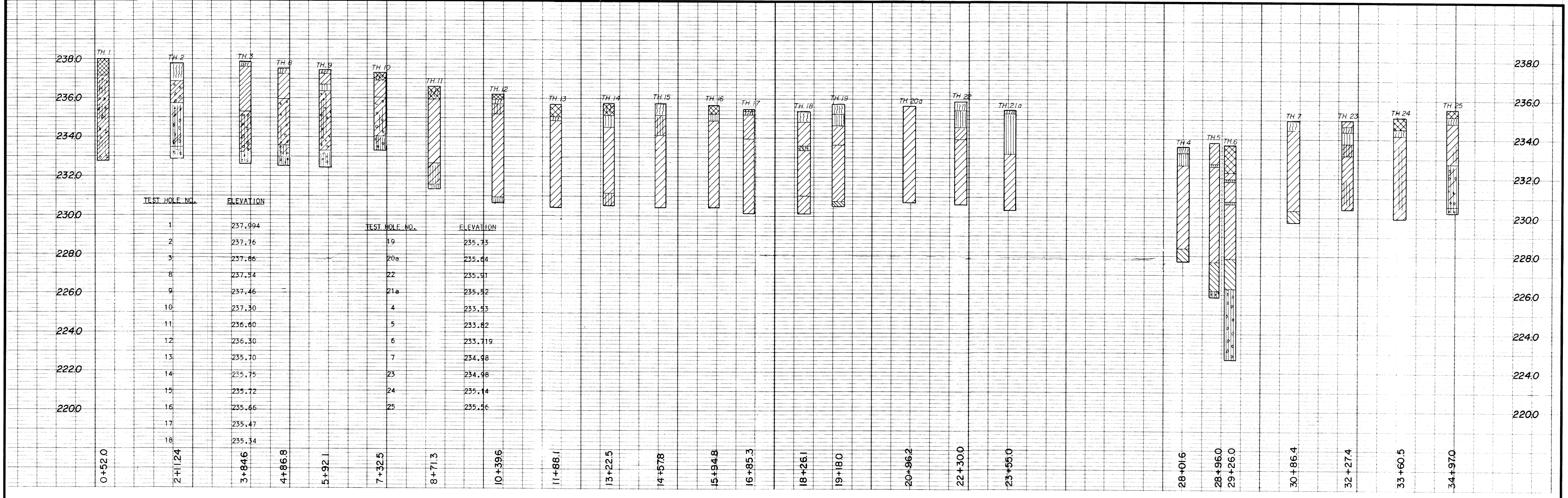
Notes:
1) Power auger refusal encountered at 16.7 m.
2) No seepage or sloughing observed.
3) Water at 3.0 m
4) Test hole was backfilled with auger cuttings 0.5 m bentonite at bottom of test hole and 0.5 m bentonite at top
5) Test hole was open to 10.7 m

Logged By: Syl Precourt

Reviewed By: Michael Van Helden

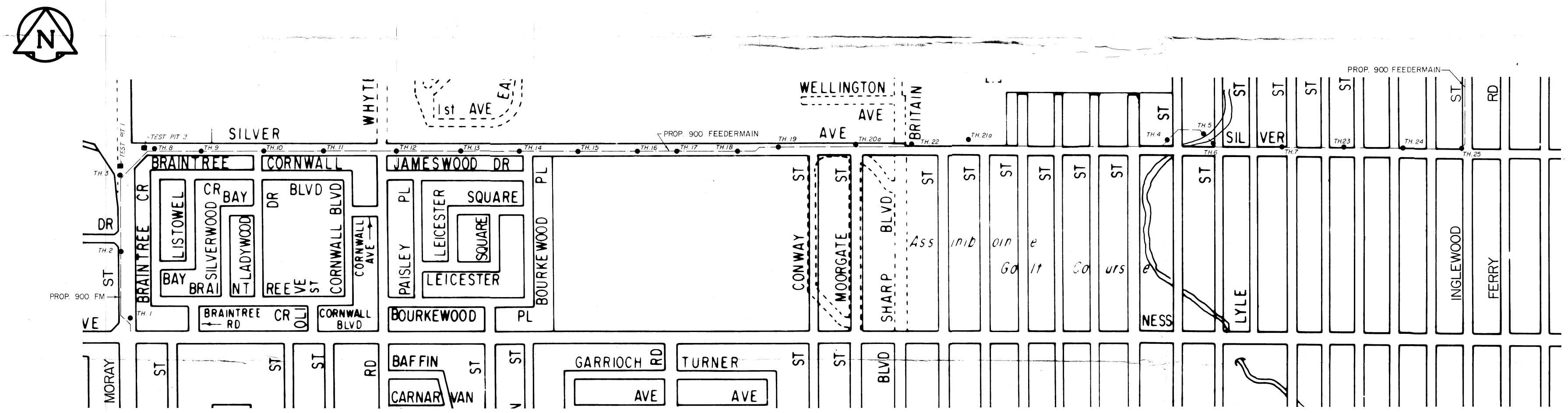
Project Engineer: Michael Van Helden

WO 102-84



TEST HOLE NO.	ELEVATION
1	237.994
2	237.76
3	237.86
8	237.54
9	237.46
10	237.30
11	236.60
12	236.30
13	235.70
14	235.75
15	235.72
16	235.66
17	235.47
18	235.34

TEST HOLE NO.	ELEVATION
19	235.73
20a	235.64
22	235.91
21a	235.52
4	233.53
5	233.82
6	233.719
7	234.98
23	234.98
24	235.14
25	235.56



LEGEND

▥	TOPSOIL
☒	FILL
▨	CLAY
▥	SILT
⁘	TILL

B.M. ELEV.			
NO.	REVISIONS	DATE	BY

WORKS & OPERATIONS DIVISION
WATERWORKS, WASTE & DISPOSAL DEPARTMENT

DESIGNED BY	CHECKED BY TH
DRAWN BY RN	APPROVED BY
HOR. SCALE 1:5000 VERTICAL 1:100	RELEASED FOR CONSTRUCTION
DATE JULY, 1987	DATE

ENGINEER'S SEAL
R. J. WITTEBOLLE

CONSULTANT DRAWING NO.

THE CITY OF WINNIPEG
WORKS AND OPERATIONS DIVISION

WEST END FEEDERMAIN

CITY DRAWING NUMBER

SHEET OF

01

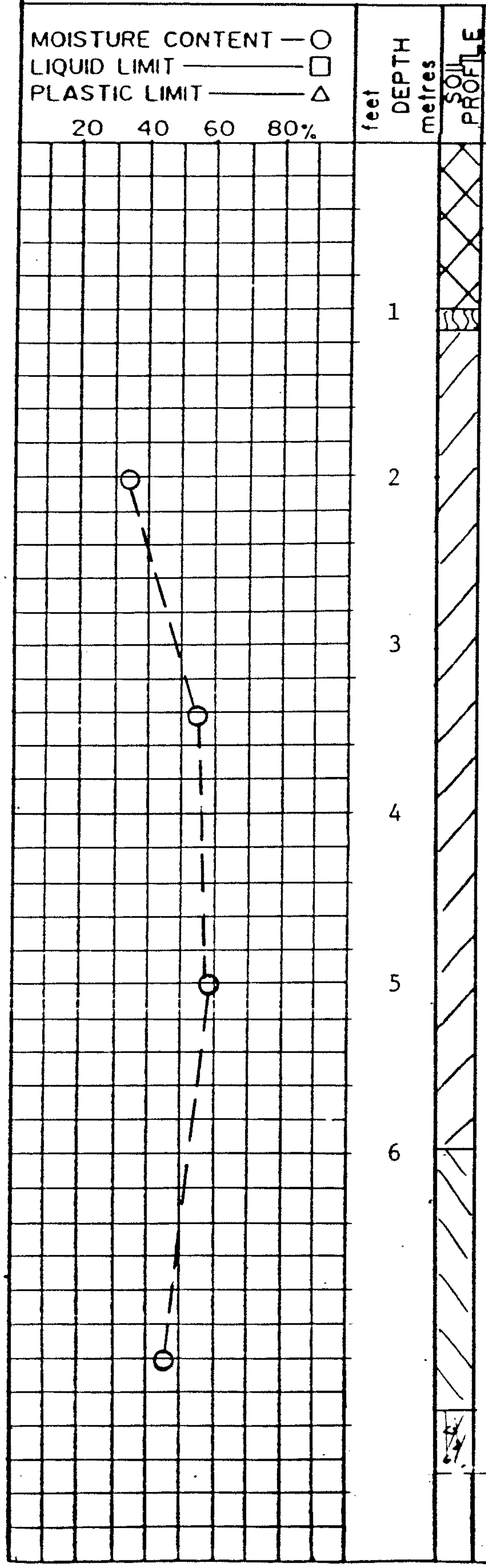
UMA Engineering Ltd.
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 16, 1986
DRILLED BY: BM DRILLING LTD.

TEST HOLE NO. 5

MOISTURE CONTENT — ○
LIQUID LIMIT — □
PLASTIC LIMIT — △

SURFACE ELEVATION: 233.82m
CO-ORDINATES:



SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
FILL - clay and stones - dry - stiff				
CLAY (topsoil) - black - organic - damp - stiff				
CLAY - brown - trace of silt - plastic - weathered in upper portion - stiff	B4			γ_d=13.31 KN/m^3 γ_w=17.99 KN/m^3 L_v=86.8 kPa
	B5			L_v=57.6 kPa
	B6			γ_d=10.46 KN/m^3 γ_w=16.57 KN/m^3 L_v=56.6 kPa
CLAY - grey - occasional till inclusions - firm to soft with depth	B7			γ_d=12.07 KN/m^3 γ_w=17.5 KN/m^3 L_v=39.0 kPa

End of hole at 7.9 m.

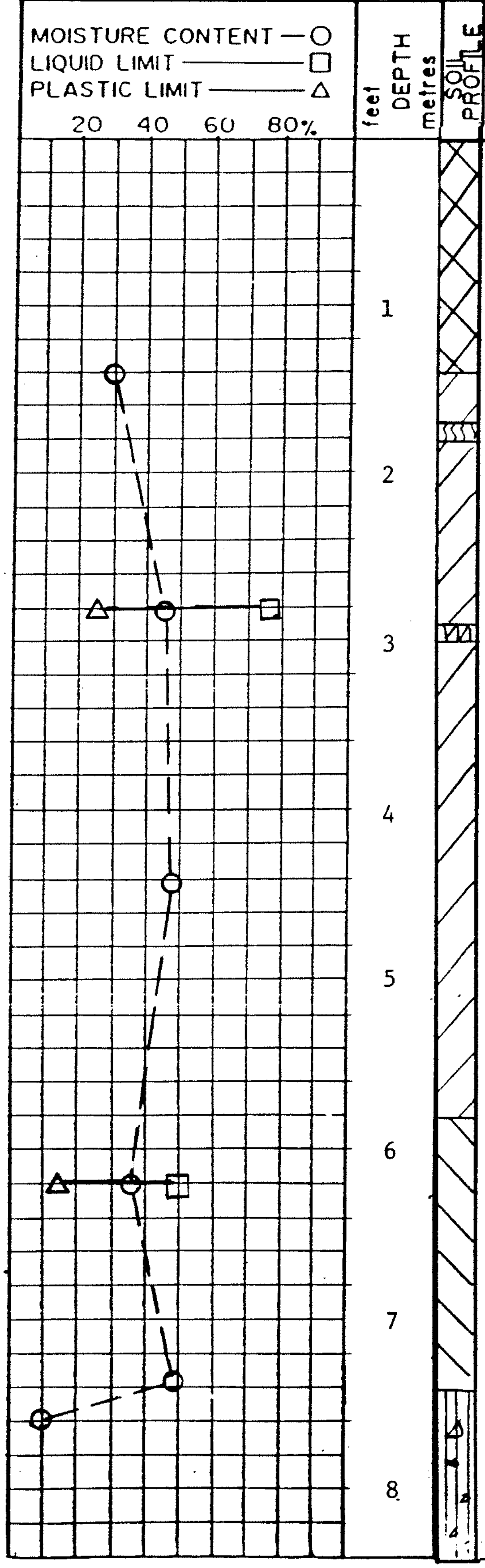
NOTES:
- no seepage during drilling.

UMA Engineering Ltd.
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 5, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 6

SURFACE ELEVATION: 233.72m
CO-ORDINATES:



SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
FILL - clay and silt - concrete and asphalt pieces - gravel - dry - stiff				
CLAY - brown - plastic - stiff to firm	1B			γ_d=13.94 KN/m^3 γ_w=18.10 KN/m^3 L_v=98.7 kPa
CLAY (topsoil) - black - organic				
CLAY - brown - plastic - trace of silt and sulphates - silty layer 2.9 - 3.0 m - stiff to firm at 3.0 m	2B			γ_d=11.92 KN/m^3 γ_w=17.42 KN/m^3 PI=49.7% L_v=56.4 kPa
	3B			γ_d=11.59 KN/m^3 γ_w=17.18 KN/m^3 L_v=65.6 kPa
CLAY - grey - occasional silt pockets and till inclusions - firm to soft with depth	4B			PI=33.8% γ_d=13.18 KN/m^3 γ_w=18.05 KN/m^3 L_v=45.2 kPa
SILT (till) - wet - soft	5B			

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UMA Engineering Ltd.
Engineers & Planners
1479 Buffalo Place, Winnipeg, Manitoba, Canada R3T 1L7

PROJECT: WEST END FEEDERMAIN
CLIENT: CITY OF WINNIPEG
JOB NO.: 0265-238-01-02
DRILLING DATE: DECEMBER 5, 1986
DRILLED BY: SUBTERRANEAN LTD.

TEST HOLE NO. 6
Contin.

MOISTURE CONTENT — ○
LIQUID LIMIT — ☐
PLASTIC LIMIT — △
20 40 60 80%

SURFACE ELEVATION: 233.72m
CO-ORDINATES:

DEPTH feet / metres	SOIL DESCRIPTION	SAMPLE NO.	STANDARD PEN.(N)	COMP. STRENGTH ☐psf ☐kPa	MISC TESTS AND REMARKS
					13.5% Clay 36.5% Silt 33% Sand 17% Gravel
9	- becoming dryer and denser at 8.5 m with cobbles				
10		7G			
11	Auger regusal at 11.0 m.				
12	NOTES: - no seepage during drilling.				
13					
14					
15					
16					

Appendix C

Visual Field Inspection Photos

Site 4 - Western Riverbank Ground between bridges gently sloping towards river (facing E)

Site 4 - Western Riverbank Steepened slopes around siphons inlet chamber structure (facing E)

Site 4 - Western Riverbank Gently sloping riverbank crest covered in brush, shrubs, and tree clusters (facing E)

Site 4 - Western Riverbank Gently sloping riverbank crest to the south of the crossing alignment(facing SE)

Site 4 - Western Riverbank	Gently sloping riverbank crest to the north of the crossing alignment (facing NE)

Site 4 - Western Riverbank	Asphalt paved pedestrian pathway. Minor cracking observed parallel to bank crest (facing S)

Site 4 - Western Riverbank	Densely vegetated riverbank crest to the east of the pedestrian pathway (facing E)

Site 4 - Western Riverbank	South bridge pier near river edge surrounded in rip-rap armouring (facing S)

Site 4 - Western Riverbank	Observed scarp near oversteepened riverbank crest in adjacent to crossing alignment (facing N)

Site 4 - Western Riverbank	Short erosion scarps, localized rip-rap, gradual toe slope within crossing alignment (facing N)

Site 4 - Western Riverbank	Short erosion scarps, localized rip-rap, gradual toe slope adjacent to crossing alignment (facing S)

Site 4 - Western Riverbank	Generally vertical oriented trees near riverbank crest (facing S)

Site 4 - Eastern Riverbank	Steeper slopes around hydro tower showed signs of slope instability and animal burrows (facing E)

Site 4 - Eastern Riverbank	Ground between bridges gently sloping towards river (facing W)

Site 4 - Eastern Riverbank	Gently sloping riverbank crest west of siphons inlet chamber structure (facing W)

Site 4 - Eastern Riverbank	Animal burrows observed in front of siphons inlet chamber structure (facing W)

Site 4 - Eastern Riverbank	Gradually sloping riverbank crest east of pedestrian pathway, groundwater well (facing S)

Site 4 - Eastern Riverbank	Gradual riverbank crest slopes east of pedestrian pathway (facing N)

Site 4 - Eastern Riverbank	Asphalt paved pedestrian pathway. Minor cracking observed parallel to bank crest (facing N)

Site 4 - Eastern Riverbank	Brush and shrubs observed along riverbank crest west of pedestrian pathway (facing W)

Site 4 - Eastern Riverbank	Riverbank slightly steepening west of pedestrian pathway, groundwater well (facing S)

Site 4 - Eastern Riverbank	Riverbank slightly steepening east of pedestrian pathway, tree clusters (facing N)

Site 4 - Eastern Riverbank	Rip-rap armouring around south bridge pier and along gradually sloping bank toe (facing S)

Site 4 - Eastern Riverbank	Rip-rap armouring along entire lower portion of riverbank between bridges (facing N)

Site 5 - Northern Riverbank	View of northern bank from top of bridge (facing NE)

Site 5 - Northern Riverbank	Gradually sloping ground down Oxbow Bend Rd. towards river (facing S)

Site 5 - Northern Riverbank	View from riverbank crest along approximate crossing alignment (facing S)

Site 5 - Northern Riverbank	Granular road along riverbank crest below bridge, jersey barriers, traffic signs (facing W)

Site 5 - Northern Riverbank Slightly steepening bank slope down towards river within eastern portion of study area (facing E)

Site 5 - Northern Riverbank Flattened bank slope near top of erosion scarp within eastern portion of study area (facing E)

Site 5 - Northern Riverbank Erosion scarp observed near bank toe within eastern portion of study area (facing E)

Site 5 - Northern Riverbank Slightly steepened bank slope down towards river within western portion of study area (facing W)

Site 5 - Northern Riverbank	Erosion scarp observed near bank toe within western portion of study area (facing W)

Site 5 - Northern Riverbank	Rip-rap along slope within discharge path of CSP outfall in western portion of study area (facing W)

Site 5 - Northern Riverbank	CSP outfall daylighting along bank slope, some erosion of bank material between rip-rap (facing N)

Site 5 - Northern Riverbank	Traffic signs located along bank crest near crossing alignment. One leaning, one straight (facing W)

Site 5 - Northern Riverbank	Concrete drainage culvert beneath roadway near bank crest close to bridge structure (facing N)

Site 5 - Southern Riverbank	View of southern bank from top of bridge (facing SE)

Site 5 - Southern Riverbank	Rock drains installed within steeper slopes of rip-rap lined drainage channel (facing N)

Site 5 - Southern Riverbank	No observed movement of lift station located at east crest of rip-rap drainage channel (facing E)

Site 5 - Southern Riverbank	Drainage channel sloped towards CSP culverts west of crossing alignment (facing NW)

Site 5 - Southern Riverbank	Discharge path of CSP culverts west of crossing alignment, gradual bank slopes (facing NW)

Site 5 - Southern Riverbank	View from riverbank crest along approximate crossing alignment (facing N)

Site 5 - Southern Riverbank	Gradual slopes, brush, shrubs, and trees observed along bank crest near crossing alignment (facing E)

Site 5 - Southern Riverbank	Flattened bank crest slope closer to river edge, signs of pedestrian passage (facing E)

Site 5 - Southern Riverbank	Gradual slopes, brush, trees observed along bank crest west of crossing alignment (facing W)

Site 5 - Southern Riverbank	Fallen tree in close proximity to crossing alignment and erosion scarp at river edge (facing NE)

Site 5 - Southern Riverbank	Rip-rap armouring along bank slope between CSP culverts and river edge (facing W)

Site 5 - Southern Riverbank	Sloped riverbank edge, erosion scarp, fallen tree in close proximity to crossing alignment (facing E)

Site 5 - Southern Riverbank	Increasing width of exposed bank further east from the crossing alignment (facing E)

Site 5 - Southern Riverbank	View near river edge along approximate crossing alignment (facing S)

Site 6A - Northern Bank	View from bank crest along approximate crossing alignment (facing SW)

Site 6A - Northern Bank	Flatter slopes around drain, steepening sharply towards bank crest (facing W)

Site 6A - Northern Bank	Flatter slopes around drain, steepening sharply towards bank crest (facing SE)

Site 6A - Northern Bank	Oversteepened bank slopes, leaning trees, brush, shrub vegetation near bank crest (facing NW)

Site 6A - Northern Bank	Scarps from slope instabilities observed along oversteepened portion of banks (facing NW)

Site 6A - Northern Bank	Consistently sloping ground from crest to bank toe east of crossing alignment (facing NW)

Site 6A - Northern Bank	Scarps observed near flatter portion near drain in vicinity of crossing alignment (facing W)

Site 6A - Northern Bank	Erosion scarp observed along drain edges, varying in height (facing W)

Site 6A - Southern Bank Flatter slopes around drain, steepening sharply towards bank crest (facing E)

Site 6A - Southern Bank Consistently sloping ground from crest to bank toe east of crossing alignment (facing E)

Site 6A - Southern Bank Progressive slope instabilities observed in close proximity to crossing alignment (facing W)

Site 6A - Southern Bank Progressive slope instabilities have progressed towards the bank crest (facing W)

Site 6A - Southern Bank	Progressive slope instabilities have progressed towards the bank crest (facing E)

Site 6A - Southern Bank	Slope instability ridges observed near bank crest west of the crossing alignment (facing W)

Site 6A - Southern Bank	Progressive slope instabilities along bank slope near crossing alignment (facing SE)

Site 6A - Southern Bank	Shallow slope instabilities observed at localized areas along bank toe (facing S)

Site 6B - Western Bank	View of western riverbank from eastern riverbank near crossing alignment (facing W)

Site 6B - Western Bank	Flatter slopes, dense brush large trees along bank crest north of crossing (facing N)

Site 6B - Western Bank	Flatter slopes steepening slightly near river, dense brush along bank crest south of crossing (facing S)

Site 6B - Western Bank	Minor erosion observed at localized areas along bank toe (facing N)

Site 6B - Western Bank	Minor erosion observed at localized areas along bank toe (facing S)

Site 6B - Eastern Bank	View of eastern riverbank from western riverbank near crossing alignment (facing E)

Site 6B - Eastern Bank	Slopes steepening slightly near river, dense brush within southern portion of study area (facing S)

Site 6B - Eastern Bank	Slightly steepening bank slope down towards river within northern portion of study area (facing E)

Site 6B - Eastern Bank	Steepened banks slope extends from bank crest down to bank toe (facing N)

Site 6B - Eastern Bank Minor erosion observed at localized areas along bank toe (facing N)

Site 6B - Eastern Bank Minor erosion observed at localized areas along bank toe (facing S)

Site 6B - Eastern Bank Animal burrows observed within the steeper bank slopes (facing E)

Site 6B - Eastern Bank Bank slopes flatten out near the river edge north of the study area (facing N)

Site 7 - Western Bank	Sturgeon Creek Greenway Trail and gradual river-bank slopes east of crossing (facing SE)

Site 7 - Western Bank	View from the west bank facing the east bank along the approximate crossing alignment (facing E)

Site 7 - Western Bank	Gradual slope, manicured grass, wood posts along riverbank crest beside bridge abutment (facing W)

Site 7 - Western Bank	Western bridge abutment near bank crest (facing N)

Site 7 - Western Bank	Cracks around MTS manhole located in paved bridge sidewalk near abutment

Site 7 - Western Bank	Steeper slope around bridge abutment and minor cracking along pedestrian pathway (facing NE)

Site 7 - Western Bank	Grouted rip-rap armouring along steeper banks in close proximity to bridge abutment (facing N)

Site 7 - Western Bank	Cracks observed within grouted rip-rap armoring at various orientations

Site 7 - Western Bank	Grouted rip-rap along abutment head slope below bridge structure (facing NW)

Site 7 - Western Bank	Exposed grouted rip-rap and brush vegetation east of pathway near crossing alignment (facing S)

Site 7 - Western Bank	Brush vegetation along bank slope near creek edge within southern portion of study area (facing N)

Site 7 - Western Bank	Localized scarps and gulley areas along exposed bank toe in southern portion of study area (facing N)

Site 7 - Western Bank	Ground sloping southeastward from the bridge structure, vertical light post (facing E)

Site 7 - Eastern Bank	View from the east bank facing the west bank along the approximate crossing alignment (facing W)

Site 7 - Eastern Bank	Steeper bank slopes close to bridge structure, under-bridge pedestrian pathway (facing W)

Site 7 - Eastern Bank	Near flat slopes and manicured grass within southern portion of study area (facing SE)

Site 7 - Eastern Bank	Brush and shrubs near bank edge within southern portion of study area (facing S)

Site 7 - Eastern Bank	Steeper slopes to the east of pedestrian pathway, gradual slope to the west of it (facing NW)

Site 7 - Eastern Bank	Grouted rip-rap armouring along steeper banks in close proximity to bridge abutment (facing N)

Site 7 - Eastern Bank	Exposed grouted rip-rap and brush vegetation west of pathway near crossing alignment (facing W)

Site 7 - Eastern Bank	Grouted rip-rap along abutment head slope below bridge structure (facing N)

Site 7 - Eastern Bank	Bank toe within southern portion of study area, indicating higher than usual water level (facing S)

Site 7 - Eastern Bank	Bank toe within southern portion of study area, indicating higher than usual water level (facing NW)

Site 7 - Eastern Bank	Beaver den observed across the creek near the bank edge (facing W)

Site 7 - Eastern Bank	Beaver dam south of study area causing higher water levels within the study area (facing W)

Site 8 - Western Bank	View of western riverbank from eastern riverbank within study area (facing NW)

Site 8 - Western Bank	Date of construction cast into Saskatchewan Ave. bridge wingwall (facing N)

Site 8 - Western Bank	Regraded and rip-rap armoured slope within crossing alignment (facing S)

Site 8 - Western Bank	Regraded and rip-rap armoured slope near bridge structure. Steeper slope near abutment (facing N)

Site 8 - Western Bank	Gradual bank slopes and dense brush and shrub coverage observed south of rip-rap (facing S)

Site 8 - Western Bank	Partially grasses bank crest between Empress St. and the bank slope (facing S)

Site 8 - Eastern Bank	View of eastern riverbank from western riverbank within study area (facing NE)

Site 8 - Eastern Bank	Approximately vertical fenceline along adjacent private property east of crossing (facing S)

Site 8 - Eastern Bank	Regraded and rip-rap armoured slope within crossing alignment (facing S)

Site 8 - Eastern Bank	Regraded and rip-rap armoured slope near bridge structure (facing N)

Site 8 - Eastern Bank	Brush and trees along riverbank crest within southern portion of study area (facing S)

Site 8 - Eastern Bank	Animal burrows observed along bank slopes.

Site 8 - Eastern Bank	Scarp ridge observed near bank crest at oversteepened bank south of rip-rap area (facing S)

Site 8 - Eastern Bank	Oversteepened banks observed within southern portion of the study area (facing N)

Site 8 - Eastern Bank	Minimal erosion observed along bank toe south of the rip-rap area (facing N)

Site 8 - Eastern Bank	Observed bank slope change due to regrading near start of rip-rap area (facing N)

Site 9 - Western Bank View of western riverbank from pedestrian bridge north of study area (facing W)

Site 9 - Western Bank Displaced rip-rap and exposed geotextile at bridge abutment north of the crossing (facing NW)

Site 9 - Western Bank Gradual slopes down from bank crest to toe, heavily damaged fence (facing SW)

Site 9 - Western Bank Moderate to dense brush vegetation along bank slope, groundwater well near bridge (facing N)

Site 9 - Western Bank Groundwater well near west bridge abutment containing pneumatic piezometer

Site 9 - Western Bank Animal burrows observed within bank slopes.

Site 9 - Western Bank Relatively flat bank crest (Assiniboine Golf Course) becoming steeper towards creek (facing SW)

Site 9 - Western Bank Relatively flat bank crest with manicured grass (Assiniboine Golf Course (facing N)

Site 9 - Eastern Bank View of eastern riverbank from pedestrian bridge north of study area (facing S)

Site 9 - Eastern Bank Gradual bank slopes densely vegetated with brush, shrubs, and trees (facing N)

Site 9 - Eastern Bank Rip-rap at bridge abutment north of the crossing (facing NE)

Site 9 - Eastern Bank Dense vegetation along bank slopes near creek (facing W)

Site 9 - Eastern Bank	Flatter slopes and manicured grass along bank crest, traffic signage (facing SW)

Site 9 - Eastern Bank	N-S portion of Silver Avenue, no significant cracks observed, generally flat bank crest (facing N)

Site 10 - Northern Riverbank	View of northern bank from southern bank along approximate crossing alignment (facing N)

Site 10 - Northern Riverbank	Bank slope located near edge of pedestrian pathway within study area (facing S)

Site 10 - Northern Riverbank	Pedestrian pathway with minor cracking and railing along bank slope (facing SW)

Site 10 - Northern Riverbank	Slope that flattens out closer to the river edge within southern portion of study area (facing E)

Site 10 - Northern Riverbank	Slope from pathway down towards river edge within northern portion of study area (facing W)

Site 10 - Northern Riverbank	Lower bank slope within northern portion of study area (facing W)

Site 10 - Northern Riverbank	Lower bank slope within southern portion of study area (facing E)

Site 10 - Northern Riverbank	Scarp near river edge observed along full length of bank toe within study area (facing W)

Site 10 - Northern Riverbank	Scarp near river edge observed along full length of bank toe within study area (facing E)

Site 10 - Northern Riverbank	Masonry retaining wall structure near pedestrian pathway shows small signs of movement (facing W)

Site 10 - Southern Riverbank View of southern bank from eastern bank along approximate crossing alignment (facing S)

Site 10 - Southern Riverbank Gradually sloped bank crest and clearing down towards river along pipe alignment (facing N)

Site 10 - Southern Riverbank Riverbank crest begins to slope more steeply closer to the river (facing N)

Site 10 - Southern Riverbank Oversteepened banks and instabilities observed within eastern portion of study area (facing E)

Site 10 - Southern Riverbank	Scarp face observed along oversteepened slope within eastern portion of study area

Site 10 - Southern Riverbank	Larger scarps and leaning trees observed along banks in eastern portion of study area (facing SE)

Site 10 - Southern Riverbank	Scarp near river edge observed within southern portion of study area (facing E)

Site 10 - Southern Riverbank	Gradually sloping bank crest within western portion of study area (facing W)

Site 10 - Southern Riverbank	Scarp near river edge observed within eastern portion of study area (facing W)

Site 10 - Southern Riverbank	Local rip-rap observed along the bank toe near the crossing alignment (facing W)

Site 10 - Southern Riverbank	Scarp near river edge observed within western portion of study area (facing W)

Site 10 - Southern Riverbank	Small scarp and crack observed along flat portion of bank crest near crossing alignment (facing S)

Appendix D

Site Reconnaissance Summary, SCG and ECG Values

APPENDIX D - SUMMARY OF VISUAL FIELD INSPECTION AND ASSIGNED SCG AND ECG RATINGS

SITE INFORMATION			PIPE ASSET			SOIL TYPE				SCARP PRESENT ON ALIGNMENT		SCARP PRESENT IN NEIGHBOURING AREAS		BANK CREST INSTABILITIES		BANK SLOPE INSTABILITIES		TOE EROSION		RIP RAP AT BANK TOE		IF RIP RAP EXISTS, COVERAGE EXTENDS SUFFICIENT DISTANCE AWAY FROM CROSSING		BRIDGE ADJACENT TO CROSSING		ASSIGNED RATING (1 TO 5) (1 - DEFECT FREE) (5 - FAILED OR FAILING)		
NAME	WATER CROSSING	NEIGHBOURING STREET(S)	PIPE DIAMETER (mm)	PIPE MATERIAL	BANK	EXISTING TH INFO AVAILABLE	ALLUVIAL	GLACIOLACUSTRINE	BOTH ALLUVIAL AND GLACIOLACUSTRINE	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	YES	NO	EXIST	NOT EXIST	SCG	ECG	COMMENTS
Site 4 - Fort Garry/St. Vital Interceptor Siphons	Red River	Bishop Grandin Boulevard	700	HDPE	West	YES			X	X		X		X			X	X		X			X	X		3	2	Evidence of shallow instabilities noted near bank crest. Rip-rap appears to be effective, but is localized to a small area around the pipe crossing alignment. Erosion into banks observed around rip-rap-armoured area. Previous stability analyses indicate FS for slip surface engaging siphons to be less than design criteria. Flagged for slope stability analysis
					East	YES			X		X		X		X		X		X	X		X		X		1	2	Some erosion observed along bank slope above rip-rap armoured area. Bank underwent slope stabilization (regrading, rip-rap toe armouring) in 2013, and slope stability analyses completed as part of these works indicate FS for slip surface engaging siphons meets design criteria. Design is consistent with site observations.
			800	HDPE	West	YES			X	X		X		X			X	X		X			X	X		3	2	Evidence of shallow instabilities noted near bank crest. No deep-seated slope instabilities observed. Rip-rap appears to be effective, but is localized to a small area around the pipe crossing alignment. Erosion into banks observed around rip-rap-armoured area. Previous stability analyses indicate FS for slip surface engaging siphons to be less than design criteria. Flagged for slope stability analysis
					East	YES			X		X		X		X		X		X	X		X		X		1	2	Some erosion observed along bank slope above rip-rap armoured area. Bank underwent slope stabilization (regrading, rip-rap toe armouring) in 2013, and slope stability analyses completed as part of these works indicate FS for slip surface engaging siphons meets design criteria. Design is consistent with site observations.
Site 5 - West Perimeter Force Main	Assiniboine River	Perimeter Highway, Oxbow Bend Road	400	Steel	North	YES			X	X		X			X		X	X			X		X	X		2	2	Feeder main installed within glacial till, and is unlikely to be intercepted by slip surface with FS below design criteria.Erosion observed near river edge, rip-rap not present within crossing alignment.
					South	YES		X		X		X			X		X	X			X		X	X		2	2	Feeder main installed within glacial till, and is unlikely to be intercepted by slip surface with FS below design criteria.Erosion observed near river edge, rip-rap not present within crossing alignment.
Site 6A - Dakota Feeder Main	Navin Drain	Bishop Grandin Boulevard	600	PCCP	North	NO				X		X		X		X		X			X		X		X	2	2	Pipe buried deep within the banks at this site, and unlikely to be engaged by slip surfaces with FS less than design criteria. Instabilities due to oversteepened banks and erosion observed do not pose a short-term risk to the pipe crossing.
					South	NO				X		X		X		X		X			X		X		X	2	2	Pipe buried deep within the banks at this site, and unlikely to be engaged by slip surfaces with FS less than design criteria. Instabilities due to oversteepened banks and erosion observed do not pose a short-term risk to the pipe crossing.
Site 6B - Dakota Feeder Main	Seine River	Bishop Grandin Boulevard	600	PCCP	West	NO					X		X		X		X	X			X		X		X	1	2	Slope beyond bank crest very gradual. Erosion observed near river edge, rip-rap not present within crossing alignment.
					East	NO					X		X		X		X	X			X		X		X	1	2	Erosion observed near river edge, rip-rap not present within crossing alignment

APPENDIX D - SUMMARY OF VISUAL FIELD INSPECTION AND ASSIGNED SCG AND ECG RATINGS

SITE INFORMATION			PIPE ASSET			SOIL TYPE				SCARP PRESENT ON ALIGNMENT		SCARP PRESENT IN NEIGHBOURING AREAS		BANK CREST INSTABILITIES		BANK SLOPE INSTABILITIES		TOE EROSION		RIP RAP AT BANK TOE		IF RIP RAP EXISTS, COVERAGE EXTENDS SUFFICIENT DISTANCE AWAY FROM CROSSING		BRIDGE ADJACENT TO CROSSING		ASSIGNED RATING (1 TO 5) (1 - DEFECT FREE) (5 - FAILED OR FAILING)		
NAME	WATER CROSSING	NEIGHBOURING STREET(S)	PIPE DIAMETER (mm)	PIPE MATERIAL	BANK	EXISTING TH INFO AVAILABLE	ALLUVIAL	GLACIOLACUSTRINE	BOTH ALLUVIAL AND GLACIOLACUSTRINE	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	EXIST	NOT EXIST	YES	NO	EXIST	NOT EXIST	SCG	ECG	COMMENTS
Site 7 - Rouge Road Feeder Main	Sturgeon Creek	Hamilton Avenue	600	PCCP	West	YES		X			X		X		X		X	X		X			X	X		2	2	Cracking observed within grouted rip-rap around bridge abutment. Crossing alignment near interface between armoured and non-armoured bank slope. Damming of the creek has resulted in elevated creek levels and inability to view much of the lower bank slope.
					East	NO					X		X		X		X	X		X			X	X		2	2	Cracking observed within grouted rip-rap around bridge abutment. Crossing alignment near interface between armoured and non-armoured bank slope. Damming of the creek has resulted in elevated creek levels and inability to view much of the lower bank slope.
Site 8 - West End Feeder Main	Omand's Creek	Saskatchewan Avenue, Empress Street	900	PCCP	West	YES		X			X		X		X		X		X	X		X		X		1	2	Erosion observed near creek edge south of rip-rap armoured section of bank within the study area. Bank underwent slope stabilization (regrading, rip-rap armouring) as part of bridge construction, and slope stability analyses completed as part of these works indicate FS for slip surface engaging siphons meets design criteria. Design is consistent with site observations.
					East	YES		X			X	X		X		X			X	X		X		X		2	2	Slope instabilities observed in oversteepened banks and toe erosion observed south of the rip-rap armoured portion of the bank within the study area. Bank underwent slope stabilization (regrading, rip-rap armouring) as part of bridge construction, and slope stability analyses completed as part of these works indicate FS for slip surface engaging pipe meets design criteria. Design is consistent with site observations.
Site 9 - West End Feeder Main	Truro Creek	Silver Avenue	900	PCCP	West	YES		X			X		X		X		X	X			X		X	X		1	2	Erosion observed near creek edge, rip-rap not present within crossing alignment. Slope stability analyses completed as part of the pipe crossing design indicate FS for slip surface engaging pipe meets design criteria. Design is consistent with site observations.
					East	YES		X			X		X		X		X	X			X		X	X		1	2	Erosion observed near creek edge, rip-rap not present within crossing alignment. Slope stability analyses completed as part of the pipe crossing design indicate FS for slip surface engaging pipe meets design criteria. Design is consistent with site observations.
Site 10 - Haney-Moray Feeder Main	Assiniboine River	William R. Clement Parkway	450	CPP	North	NO				X		X			X	X		X			X		X	X		2	3	Erosion scarp near river edge, rip-rap not present within crossing alignment. Subsurface conditions unknown due to absence of existing geotechnical information. Discrepancies observed between as-built records and those observed on site. Flagged for geotech investigation and slope stability analysis
					South	NO				X		X		X		X		X		X			X	X		3	3	Slope instabilities observed within eastern portion of study area and near crossing alignment. Erosion scarp near river edge, sparse rip-rap at bank toe within crossing alignment. Subsurface conditions unknown due to absence of existing geotechnical information. Discrepancies observed between as-built conditions and those observed on site. Flagged for geotech investigation and slope stability analysis

Appendix E

AECOM 2021 Geotechnical Investigation: Test Hole Location Plans

Project Management Initials: Designer: Checked: Approved: ANSI A 215.9mm x 279.4mm

Last saved by: COOPERKL(2021-02-02) Last Plotted: 2021-03-16
Filename: L:\DCS\PROJECTS\WTR\60645745\900_CAD_GIS\910_CAD\25-SKETCHES\60645745-SKE-30-0000-C-10001.DWG

Issue Status: Final

HIGH RISK RIVER CROSSINGS
PHASE 3
CITY OF WINNIPEG
Project No.: 60645745 Date: 2021-03-16

Test Hole Location Plan
Site 5
West Perimeter Bridge FRM
(Assiniboine River)

AECOM

Figure: E1

Issue Status: Final

Last saved by: COOPERKL(2021-02-02) Last Plotted: 2021-03-16
Filename: L:\DCS\PROJECTS\WTR\60645745\900_CAD_GIS\910_CAD\25-SKETCHES\60645745-SKE-30-0000-C-10001.DWG
Project Management Initials: Designer: ____ Checked: ____ Approved: ____ ANSI A 215.9mm x 279.4mm

HIGH RISK RIVER CROSSINGS
PHASE 3
CITY OF WINNIPEG
Project No.: 60645745 Date: 2021-03-16

Test Hole Location Plan
Site 10
Haney-Moray FM
(Assiniboine River)

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Figure: E2

Appendix F

AECOM 2021 Geotechnical Investigation: Test Hole Logs

AECOM Canada Ltd.

GENERAL STATEMENT

NORMAL VARIABILITY OF SUBSURFACE CONDITIONS

The scope of the investigation presented herein is limited to an investigation of the subsurface conditions as to suitability for the proposed project. This report has been prepared to aid in the evaluation of the site and to assist the engineer in the design of the facilities. Our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of earth work, foundations and similar. In the event of any changes in the basic design or location of the structures as outlined in this report or plan, we should be given the opportunity to review the changes and to modify or reaffirm in writing the conclusions and recommendations of this report.

The analysis and recommendations presented in this report are based on the data obtained from the borings and test pit excavations made at the locations indicated on the site plans and from other information discussed herein. This report is based on the assumption that the subsurface conditions everywhere are not significantly different from those disclosed by the borings and excavations. However, variations in soil conditions may exist between the excavations and, also, general groundwater levels and conditions may fluctuate from time to time. The nature and extent of the variations may not become evident until construction. If subsurface conditions differ from those encountered in the exploratory borings and excavations, are observed or encountered during construction, or appear to be present beneath or beyond excavations, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Since it is possible for conditions to vary from those assumed in the analysis and upon which our conclusions and recommendations are based, a contingency fund should be included in the construction budget to allow for the possibility of variations which may result in modification of the design and construction procedures.

In order to observe compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated, we recommend that all construction operations dealing with earth work and the foundations be observed by an experienced soils engineer. We can be retained to provide these services for you during construction. In addition, we can be retained to review the plans and specifications that have been prepared to check for substantial conformance with the conclusions and recommendations contained in our report.

EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results, as shown for each hole, are described below.

1. NATURAL MOISTURE CONTENT

The relationship between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits for a sample should be compared to its natural moisture content and plotted on the Plasticity Chart in order to determine the soil classification.

2. SOIL PROFILE AND DESCRIPTION

Each soil stratum is classified and described noting any special conditions. The Modified Unified Classification System (MUCS) is used. The soil profile refers to the existing ground level at the time the hole was done. Where available, the ground elevation is shown. The soil symbols used are shown in detail on the soil classification chart.

3. TESTS ON SOIL SAMPLES

Laboratory and field tests are identified by the following and are on the logs:

N - <u>Standard Penetration Test (SPT) Blow Count</u>. The SPT is conducted in the field to assess the in-situ consistency of cohesive soils and the relative density of non-cohesive soils. The N value recorded is the number of blows from a 63.5 kg hammer dropped 760 mm which is required to drive a 51 mm split spoon sampler 300 mm into the soil.

SO_4 - <u>Water Soluble Sulphate Content</u>. Expressed in percent. Conducted primarily to determine requirements for the use of sulphate resistant cement. Further details on the water-soluble sulphate content are given in Section 6.

γ_D - <u>Dry Unit Weight</u>. Usually expressed in kN/m^3.

γ_T - <u>Total Unit Weight</u>. Usually expressed in kN/m^3.

Q_U - <u>Unconfined Compressive Strength</u>. Usually expressed in kPa and may be used in determining allowable bearing capacity of the soil.

C_U - Undrained Shear Strength. Usually expressed in kPa. This value is determined by either a direct shear test or by an unconfined compression test and may also be used in determining the allowable bearing capacity of the soil.

C_{PEN} - Pocket Penetrometer Reading. Usually expressed in kPa. Estimate of the undrained shear strength as determined by a pocket penetrometer.

The following tests may also be performed on selected soil samples and the results are given on separate sheets enclosed with the logs:

- Grain Size Analysis
- Standard or Modified Proctor Compaction Test
- California Bearing Ratio Test
- Direct Shear Test
- Permeability Test
- Consolidation Test
- Triaxial Test

4. SOIL DENSITY AND CONSISTENCY

The SPT test described above may be used to estimate the consistency of cohesive soils and the density of cohesionless soils. These approximate relationships are summarized in the following tables:

Table 1 Cohesive Soils

N	Consistency	C_u (kPa) approx.
0 - 1	Very Soft	<10
1 - 4	Soft	10 - 25
4 - 8	Firm	25 - 50
8 - 15	Stiff	50 - 100
15 - 30	Very Stiff	100 - 200
30 - 60	Hard	200 - 300
>60	Very Hard	>300

Table 2 Cohesionless Soils

N	Density
0 - 5	Very Loose
5 - 10	Loose
10 - 30	Compact
30 - 50	Dense
>50	Very Dense

5. SAMPLE CONDITION AND TYPE

The depth, type, and condition of samples are indicated on the logs by the following symbols:

■	Grab	☰	Bulk
⧄	No Recovery	▥	Shelby Tube
☒	Split Spoon	◨	Core Sample

6. WATER SOLUBLE SULPHATE CONCENTRATION

The following table, from CSA Standard A23.1-14, indicates the requirements for concrete subjected to sulphate attack based upon the percentage of water-soluble sulphate as presented on the logs. CSA Standard A23.1-14 should be read in conjunction with the table.

Table 3 Requirements for Concrete Subjected to Sulphate Attack*

Class of exposure	Degree of exposure	Water-soluble sulphate (SO_4)† in soil sample, %	Sulphate (SO_4) in groundwater samples, mg/L‡	Water soluble sulphate (SO_4) in recycled aggregate sample, %	Cementing materials to be used§††	Performance requirements§,§§		
						Maximum expansion when tested using CSA A3004-C8 Procedure A at 23 °C, %		Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % †††
						At 6 months	At 12 months††	At 18 months‡‡
S-1	Very severe	> 2.0	> 10 000	> 2.0	HS** ,HSb, HSLb*** or HSe	0.05	0.10	0.10
S-2	Severe	0.20–2.0	1500–10 000	0.60–2.0	HS**, HSb, HSLb*** or HSe	0.05	0.10	0.10
S-3	Moderate (including seawater exposure*)	0.10–0.20	150–1500	0.20–0.60	MS, MSb, MSe, MSLb***, LH, LHb, HS**, HSb, HSLb*** or HSe	0.10		0.10

*For sea water exposure, also see Clause 4.1.1.5.

†In accordance with CSA A23.2-3B.

‡In accordance with CSA A23.2-2B.

§Where combinations of supplementary cementing materials and portland or blended hydraulic cements are to be used in the concrete mix design instead of the cementing materials listed, and provided they meet the performance requirements demonstrating equivalent performance against sulphate exposure, they shall be designated as MS equivalent (MSe) or HS equivalent (HSe) in the relevant sulphate exposures (see Clauses 4.1.1.6.2, 4.2.1.1, and 4.2.1.3, and 4.2.1.4).

**Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater. See Clause 4.1.1.6.3.

††The requirement for testing at 5 °C does not apply to MS, HS, MSb, HSb, and MSe and HSe combinations made without portland limestone cement.

‡‡ If the increase in expansion between 12 and 18 months exceeds 0.03%, the sulphate expansion at 24 months shall not exceed 0.10% in order for the cement to be deemed to have passed the sulphate resistance requirement.

§§For demonstrating equivalent performance, use the testing frequency in Table 1 of CSA A3004-A1 and see the applicable notes to Table A3 in A3001 with regard to re-establishing compliance if the composition of the cementing materials used to establish compliance changes.

***Where MSLb or HSLb cements are proposed for use, or where MSe or HSe combinations include Portland-limestone cement, they must also contain a minimum of 25% Type F fly ash or 40% slag or 15% metakaolin (meeting Type N pozzolan requirements) or a combination of 5% Type SF silica fume with 25% slag or a combination of 5% Type SF silica fume with 20% Type F fly ash. For some proposed MSLb, HSLb, and MSe or HSe combinations that include Portland-limestone cement, higher SCM replacement levels may be required to meet the A3004-C8 Procedure B expansion limits. Due to the 18-month test period, SCM replacements higher than the identified minimum levels should also be tested. In addition, sulphate resistance testing shall be run on MSLb and HSLb cement and MSe or HSe combinations that include Portland-limestone cement at both 23 °C and 5 °C as specified in the table.

†††If the expansion is greater than 0.05% at 6 months but less than 0.10% at 1 year, the cementing materials combination under test shall be considered to have passed.

7. SOIL CORROSIVITY

The following table, from the Handbook of Corrosion Engineering (Roberge, 1999) indicates the corrosivity rating can be obtained from the soil resistivity, presented on the logs.

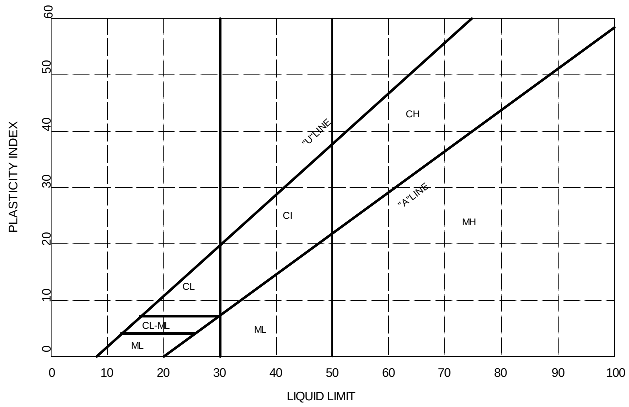
Table 4 Corrosivity Ratings Based on Soil Resistivity

Soil Resistivity (ohm-cm)	Corrosivity Rating
>20,000	Essentially non-corrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
<1,000	Extremely corrosive

8. GROUNDWATER TABLE

The groundwater table is indicated by the equilibrium level of water in a standpipe installed in a testhole or test pit. This level is generally taken at least 24 hours after installation of the standpipe. The groundwater level is subject to seasonal variations and is usually highest in the spring. The symbol on the logs indicating the groundwater level is an inverted solid triangle (▼).

MAJOR DIVISION			LOG SYMBOLS	UCS	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS	GRAVELS (MORE THAN HALF COARSE GRAINS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 4 \; C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				GP	POORLY GRADED GRAVELS AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
	SANDS (MORE THAN HALF COARSE GRAINS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE R NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}} > 6 \; C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
				SP	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		SANDS WITH FINES		SM	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW 'A' LINE W_p LESS THAN 4
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE 'A' LINE W_p MORE THAN 7
FINE GRAINED SOILS	SILTS (BELOW 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50$		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (SEE BELOW)	
		$W_L > 50$		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE 'A' LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30$		CL	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER 'F'. E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$30 < W_L < 50$		CI	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50$		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW 'A' LINE)	$W_L < 50$		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY		
		$W_L > 50$		OH	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS				Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOUR OR ODOUR, AND OFTEN FIBROUS TEXTURE	
BEDROCK				BR	SEE REPORT DESCRIPTION		
FILL				FILL	SEE REPORT DESCRIPTION		



NOTE:

1. BOUNDARY CLASSIFICATION POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%

SOIL COMPONENTS

FRACTION		SIEVE SIZE (mm)		DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS	
		PASSING	RETAINED	PERCENT	IDENTIFIER
GRAVEL	COARSE	75	19	50 - 35	AND
	FINE	19	4.75		
SAND	COARSE	4.75	2.00	35 – 20	_____Y
	MEDIUM	2.00	0.425		
	FINE	0.425	0.080	20 – 10	SOME
SILT (non-plastic) or CLAY (plastic)		0.080		10 - 1	TRACE

OVERSIZE MATERIALS

ROUNDED OR SUB-ROUNDED	ANGULAR
COBBLES 75 mm TO 200 mm BOULDERS >200 mm	ROCK FRAGMENTS ROCKS > 0.75 m3 IN VOLUME

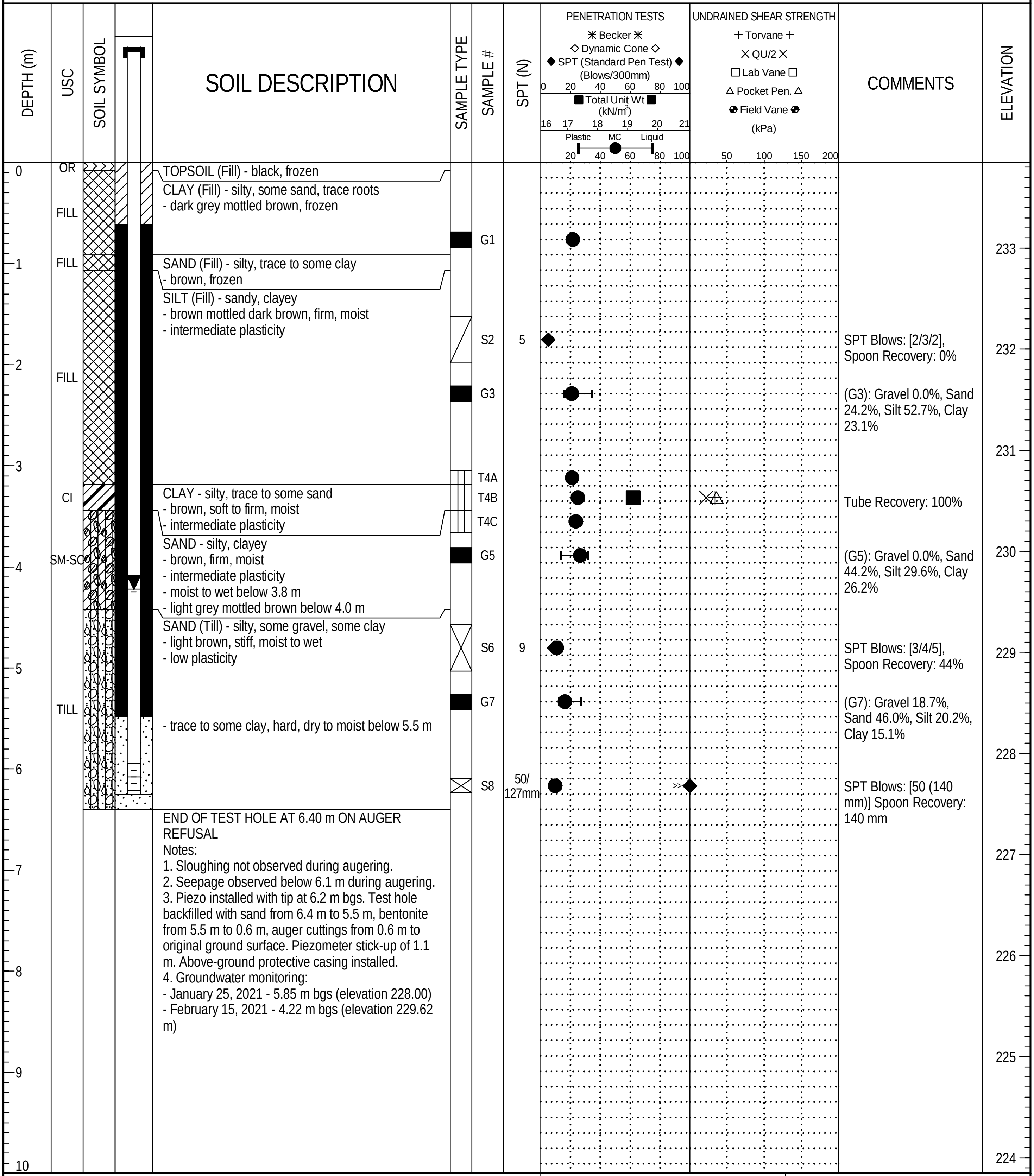
MODIFIED UNIFIED SOIL CLASSIFICATION SYSTEM

August 2015

PROJECT: High Risk River Crossing Phase 3	CLIENT: City of Winnipeg	TESTHOLE NO: TH21-01
LOCATION: Site 5 - North Bank (5525506 m N, 620343 m E)		PROJECT NO.: 60645745
CONTRACTOR: Maple Leaf Drilling	METHOD: Track-Mounted - 125 mm SSA	ELEVATION (m): 233.85

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ◿ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND



USC	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
OR	TOPSOIL (Fill) - black, frozen			
FILL	CLAY (Fill) - silty, some sand, trace roots - dark grey mottled brown, frozen	G1		
FILL	SAND (Fill) - silty, trace to some clay - brown, frozen			
FILL	SILT (Fill) - sandy, clayey - brown mottled dark brown, firm, moist - intermediate plasticity	S2	5	SPT Blows: [2/3/2], Spoon Recovery: 0%
		G3		(G3): Gravel 0.0%, Sand 24.2%, Silt 52.7%, Clay 23.1%
CI	CLAY - silty, trace to some sand - brown, soft to firm, moist - intermediate plasticity	T4A T4B T4C		Tube Recovery: 100%
SM-SC	SAND - silty, clayey - brown, firm, moist - intermediate plasticity - moist to wet below 3.8 m - light grey mottled brown below 4.0 m	G5		(G5): Gravel 0.0%, Sand 44.2%, Silt 29.6%, Clay 26.2%
TILL	SAND (Till) - silty, some gravel, some clay - light brown, stiff, moist to wet - low plasticity	S6	9	SPT Blows: [3/4/5], Spoon Recovery: 44%
		G7		(G7): Gravel 18.7%, Sand 46.0%, Silt 20.2%, Clay 15.1%
	- trace to some clay, hard, dry to moist below 5.5 m	S8	50/ 127mm	SPT Blows: [50 (140 mm)] Spoon Recovery: 140 mm

END OF TEST HOLE AT 6.40 m ON AUGER REFUSAL

Notes:

1. Sloughing not observed during augering.
2. Seepage observed below 6.1 m during augering.
3. Piezo installed with tip at 6.2 m bgs. Test hole backfilled with sand from 6.4 m to 5.5 m, bentonite from 5.5 m to 0.6 m, auger cuttings from 0.6 m to original ground surface. Piezometer stick-up of 1.1 m. Above-ground protective casing installed.
4. Groundwater monitoring:
- January 25, 2021 - 5.85 m bgs (elevation 228.00)
- February 15, 2021 - 4.22 m bgs (elevation 229.62 m)

AECOM

LOGGED BY: Ryan Harras	COMPLETION DEPTH: 6.40 m
REVIEWED BY: Elliott Drumright	COMPLETION DATE: 1/25/21
PROJECT ENGINEER: Marv McDonald	Page 1 of 1

PROJECT: High Risk River Crossing Phase 3	CLIENT: City of Winnipeg	TESTHOLE NO: TH21-02
LOCATION: Site 5 - South Bank (5525366 m N, 620351 m E)		PROJECT NO.: 60645745
CONTRACTOR: Maple Leaf Drilling	METHOD: Track-Mounted - 125 mm SSA	ELEVATION (m): 231.90

SAMPLE TYPE: ■ GRAB | ▥ SHELBY TUBE | ☒ SPLIT SPOON | ☰ BULK | ▨ NO RECOVERY | ▮ CORE

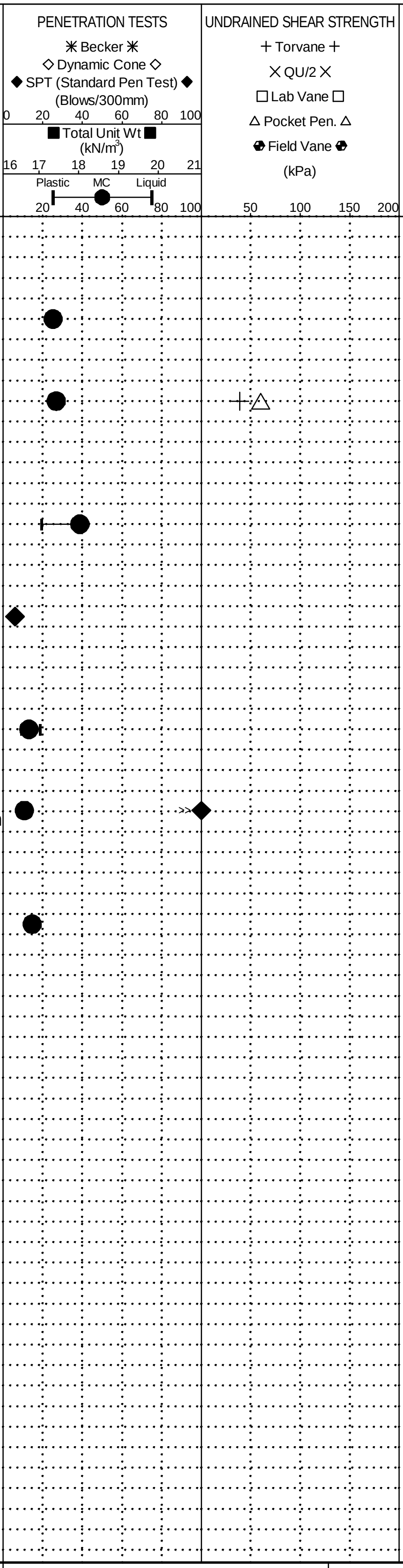
BACKFILL TYPE: ■ BENTONITE | ⁘ GRAVEL | ▥ SLOUGH | ⁘ GROUT | ▨ CUTTINGS | ⁘ SAND

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE #	SPT (N)	COMMENTS
0	OR	TOPSOIL (Fill) - black, frozen			
	FILL	CLAY (Fill) - silty, some sand, trace gravel, trace cobble, trace roots - brown, frozen to 0.9 m - firm to stiff, moist, intermediate to high plasticity below 0.9 m - cobble encountered at 1.2 m	G1 T2A		
	MI	SILT - clayey, some sand - brown mottled grey, soft to firm, moist - intermediate plasticity	T2B G3		Tube Recovery: 50% (Damaged) (G3): Gravel 0.0%, Sand 12.8%, Silt 57.5%, Clay 29.6%
	TILL	SAND (Till) - silty, some gravel, some clay - light brown, firm, moist - low plasticity - hard below 4.0 m - moist to wet below 4.6 m	S4 G5 S6 G7	6 50/ 102mm	SPT Blows: [7/3/3], Spoon Recovery: 0% SPT Blows: [26/50 (140 mm)] Spoon Recovery: 100 mm

END OF TEST HOLE AT 5.33 m ON AUGER REFUSAL

Notes:

1. Sloughing not observed during augering.
2. Seepage observed below 4.6 m during augering.
3. Piezo installed with tip at 2.4 m bgs. Test hole backfilled with bentonite from 5.3 m to 2.6 m, sand from 2.6 m to 1.8 m, and bentonite from 1.8 m to original ground surface. Piezometer stick-up of 0.9 m. Above-ground protective casing installed.
4. Groundwater monitoring:
- January 25, 2021 - 2.15 m bgs (elevation 229.75 m)
- February 15, 2021 - 2.18 m bgs (elevation 229.72 m)



DEPTH (m): 0 1 2 3 4 5 6 7 8 9 10

ELEVATION: 231 230 229 228 227 226 225 224 223 222

LOGGED BY: Ryan Harras	COMPLETION DEPTH: 5.33 m
REVIEWED BY: Elliott Drumright	COMPLETION DATE: 1/25/21
PROJECT ENGINEER: Marv McDonald	Page 1 of 1

AECOM

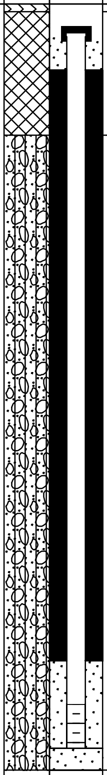
LOG OF TEST HOLE 60645745 - TEST HOLE LOGS.GPJ UMA WINN.GDT 3/16/21

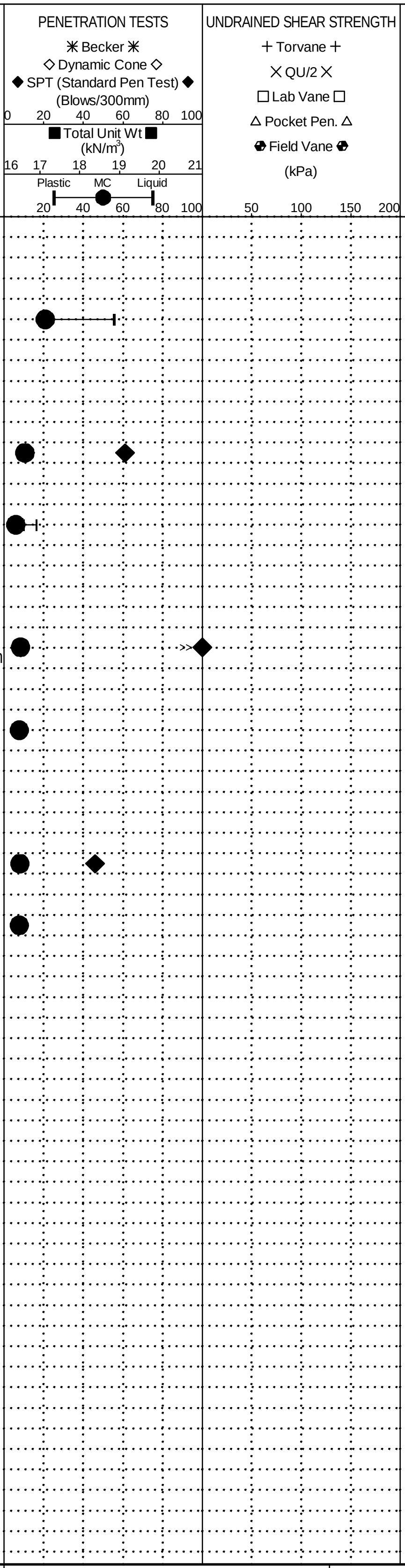
PROJECT: High Risk River Crossing Phase 3	CLIENT: City of Winnipeg	TESTHOLE NO: TH21-03
LOCATION: Site 10 - North Bank (5525903 m N, 624809 m E)		PROJECT NO.: 60645745
CONTRACTOR: Maple Leaf Drilling	METHOD: Track-Mounted - 125 mm SSA	ELEVATION (m): 231.90

SAMPLE TYPE ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ⁘ GROUT ▨ CUTTINGS ⁘ SAND

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	OR	TOPSOIL (Fill) - black, frozen					
	FILL	CLAY and SILT (Fill) - some sand, trace gravel, trace roots - dark brown, frozen - high plasticity	■	G1		(G1): Gravel 1.3%, Sand 17.8%, Silt 30.3%, Clay 50.6%	231
1		SAND and SILT (Till) - some gravel, some clay - light brown, hard, moist - low plasticity					
			☒	S2	61	SPT Blows: [12/26/35], Spoon Recovery: 33%	230
2			■	G3		(G3): Gravel 15.6%, Sand 38.6%, Silt 34.2%, Clay 11.7%	
3	TILL		☒	S4	50/ 102mm	SPT Blows: [20/50 (140 mm)], Spoon Recovery: 152 mm	229
			■	G5			228
4		- dry to moist below 4.6 m	☒	S6	46	SPT Blows: [18/21/25], Spoon Recovery: 78%	227
5			■	G7			
		END OF TEST HOLE AT 5.33 m ON AUGER REFUSAL					
6							226
7							225
8							224
9							223
10							222





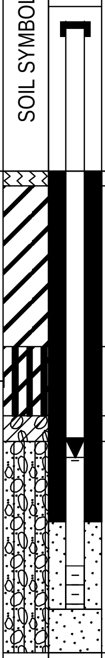
END OF TEST HOLE AT 5.33 m ON AUGER REFUSAL
Notes:
1. Sloughing not observed during augering.
2. Seepage not observed during augering.
3. Piezo installed with tip at 5.2 m bgs. Test hole backfilled with sand from 5.3 m to 4.6 m, bentonite from 4.6 m to 0.5 m, and sand from 0.5 m to 0.2 m. Flush-mount protective casing installed.
4. Groundwater monitoring:
- January 26, 2021 - Dry
- February 22, 2021 - Dry

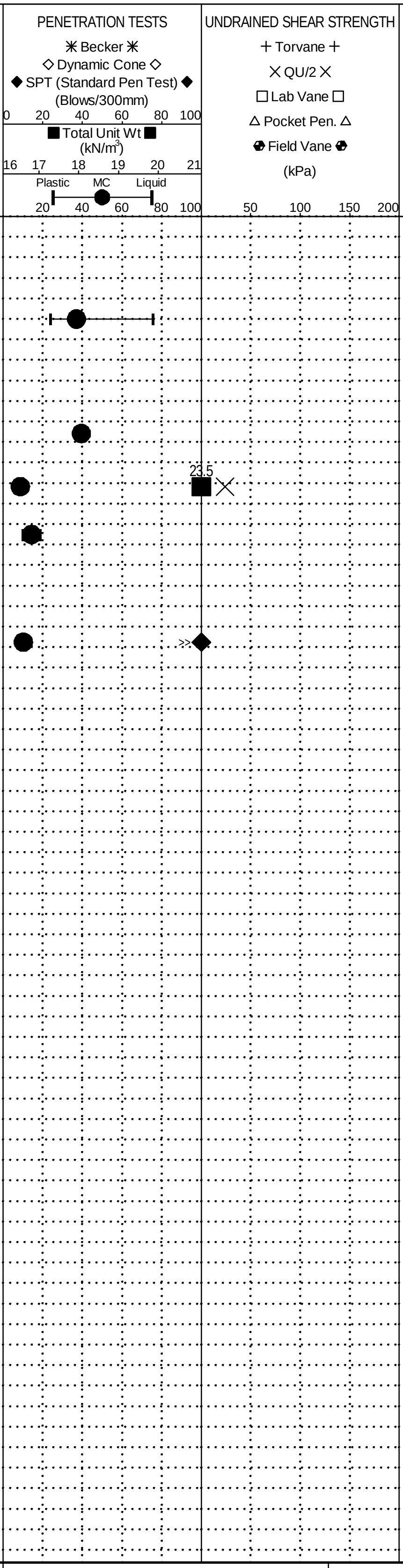
LOGGED BY: Ryan Harras	COMPLETION DEPTH: 5.33 m
REVIEWED BY: Elliott Drumright	COMPLETION DATE: 1/26/21
PROJECT ENGINEER: Marv McDonald	

PROJECT: High Risk River Crossing Phase 3	CLIENT: City of Winnipeg	TESTHOLE NO: TH21-04
LOCATION: Site 10 - South Bank (5525799 m N, 624792 m E)		PROJECT NO.: 60645745
CONTRACTOR: Maple Leaf Drilling	METHOD: Track-Mounted - 125 mm SSA	ELEVATION (m): 229.78

SAMPLE TYPE: ■ GRAB ▥ SHELBY TUBE ☒ SPLIT SPOON ☰ BULK ▨ NO RECOVERY ▮ CORE

BACKFILL TYPE: ■ BENTONITE ⁘ GRAVEL ▥ SLOUGH ▩ GROUT ▨ CUTTINGS ⁘ SAND





DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	OR	TOPSOIL (Fill) - black, frozen					
	CH	CLAY - silty, trace roots - brown, frozen to 1.1 m - high plasticity - firm, moist below 1.1 m	■	G1		(G1): Gravel 0.0%, Sand 0.3%, Silt 20.8%, Clay 78.9%	229
1	CH-MH	CLAY and SILT - some sand - grey, firm, moist - high plasticity	▥	T2A			
	SC	SAND - some clay to clayey, trace silt - grey mottled brown, firm, moist - low plasticity	▥	T2B T2C		Tube Recovery: 100%	228
2	TILL	SAND and SILT (Till) - some clay, trace gravel, trace cobble - light brown, soft, moist - low plasticity - hard below 2.3 m - suspected cobble/boulder encountered at 2.4 m during drilling	■	G3		(G3): Gravel 5.6%, Sand 38.8%, Silt 37.8%, Clay 17.8%	227
3			☒	S4	50/ 76mm	SPT Blows: [16/50 (75 mm)], Spoon Recovery: 152 mm	
		END OF TEST HOLE AT 3.35 m ON AUGER REFUSAL Notes: 1. Sloughing not observed during augering. 2. Seepage not observed during augering. 3. Suspected cobble/boulder encountered at 2.4 m during drilling. Shifted test hole by 0.2 m and re-drilled. 4. Piezo installed with tip at 3.1 m bgs. Test hole backfilled with sand from 3.4 m to 2.4 m and bentonite from 2.4 m to original ground surface. Piezometer stick-up of 1.0 m. Above-ground protective casing installed. 5. Groundwater monitoring: - January 26, 2021 - Dry - February 22, 2021 - 1.99 m bgs (elevation 227.79 m)					226
4							225
5							224
6							223
7							222
8							221
9							220
10							

AECOM

LOGGED BY: Ryan Harras	COMPLETION DEPTH: 3.35 m
REVIEWED BY: Elliott Drumright	COMPLETION DATE: 1/26/21
PROJECT ENGINEER: Marv McDonald	Page 1 of 1

Appendix G

AECOM 2021 Geotechnical Investigation: Laboratory Testing Results

AECOM
AECOM
99 Commerce Drive
Winnipeg, MB, Canada R3P 0Y7
www.aecom.com

204 477 5381 tel
204 284 2040 fax

Memorandum

To	Ryan Harras	Page	1
CC			
Subject	HRRC Phase 3 – City of Winnipeg –Test Results		
From	Elliott E. Drumright		
Date	February 18, 2021	Project Number	60645745.22

Please find attached the following material test result(s) on sample(s) submitted to the Winnipeg Geotechnical Laboratory:

- Twenty-four (24) Moisture Content Determination Test.
- Nine (9) Atterberg Limits (3 Points) test.
- Eight (8) Grain Size Distribution (Hydrometer method) test.
- Two (2) Torvane, Pocket Penetrometer, Moisture Content, Bulk Density and Visual Description with Unconfined Compressive Strength on Shelby Tube Samples.

If you have any questions, please contact the undersigned.

Sincerely,

Elliott E. Drumright

Elliott E. Drumright, Ph.D.
Associate Geotechnical Engineer

Att.

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name: HRRC Phase 3
Project Number: 60645745
Client: City of Winnipeg
Sample Location: Varies
Sample Depth: Varies
Sample Number: Varies

Supplier: AECOM
Specification: N/A
Field Technician: RHarras
Sample Date: 1/25-26/2021
Lab Technician: EManimbao
Date Tested: February 2, 2020

Moisture Content (ASTM D2216-10)

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

Location	Sample	Depth (m)	**Moisture Content (%)**	Location	Sample	Depth (m)	**Moisture Content (%)**
TH21-01	G1	0.76 - 0.91 m	**21.5%**				
	S2	1.52 - 1.98 m	**-**				
	G3	2.29 - 2.44 m	**20.8%**				
	T4A	3.05 - 3.19 m	**21.0%**				
	T4B	3.19 - 3.44 m	**24.8%**				
	T4C	3.44 - 3.66 m	**23.5%**				
	G5	3.81 - 3.96 m	**26.4%**				
	S6	4.57 - 5.03 m	**10.7%**				
	G7	5.33 - 5.49 m	**16.2%**				
	S8	6.10 - 6.55 m	**-**				
TH21-02	G1	0.76 - 0.91 m	**25.2%**				
	T2	1.22 - 1.83 m	**26.8%**				
	G3	2.29 - 2.44 m	**38.7%**				
	S4	2.74 - 3.20 m	**-**				
	G5	3.81 - 3.96 m	**13.0%**				
	S6	4.27 - 4.72 m	**-**				
	G7	5.33 - 5.49 m	**14.7%**				
TH21-03	G1	0.76 - 0.91 m	**20.8%**				
	S2	1.52 - 1.98 m	**10.5%**				
	G3	2.29 - 2.44 m	**5.9%**				
	S4	3.05 - 3.51 m	**8.3%**				
	G5	3.81 - 3.96 m	**7.7%**				
	S6	4.57 - 5.03 m	**8.0%**				
	G7	5.33 - 5.49 m	**7.7%**				
TH21-04	G1	0.76 - 0.91 m	**37.0%**				
	T2A	1.52 - 1.70 m	**39.5%**				
	T2B	1.70 - 1.88 m	**-**				
	T2C	1.88 - 2.13 m	**8.7%**				
	G3	2.29 - 2.44 m	**14.4%**				
	S4	3.05 - 3.51 m	**-**				

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

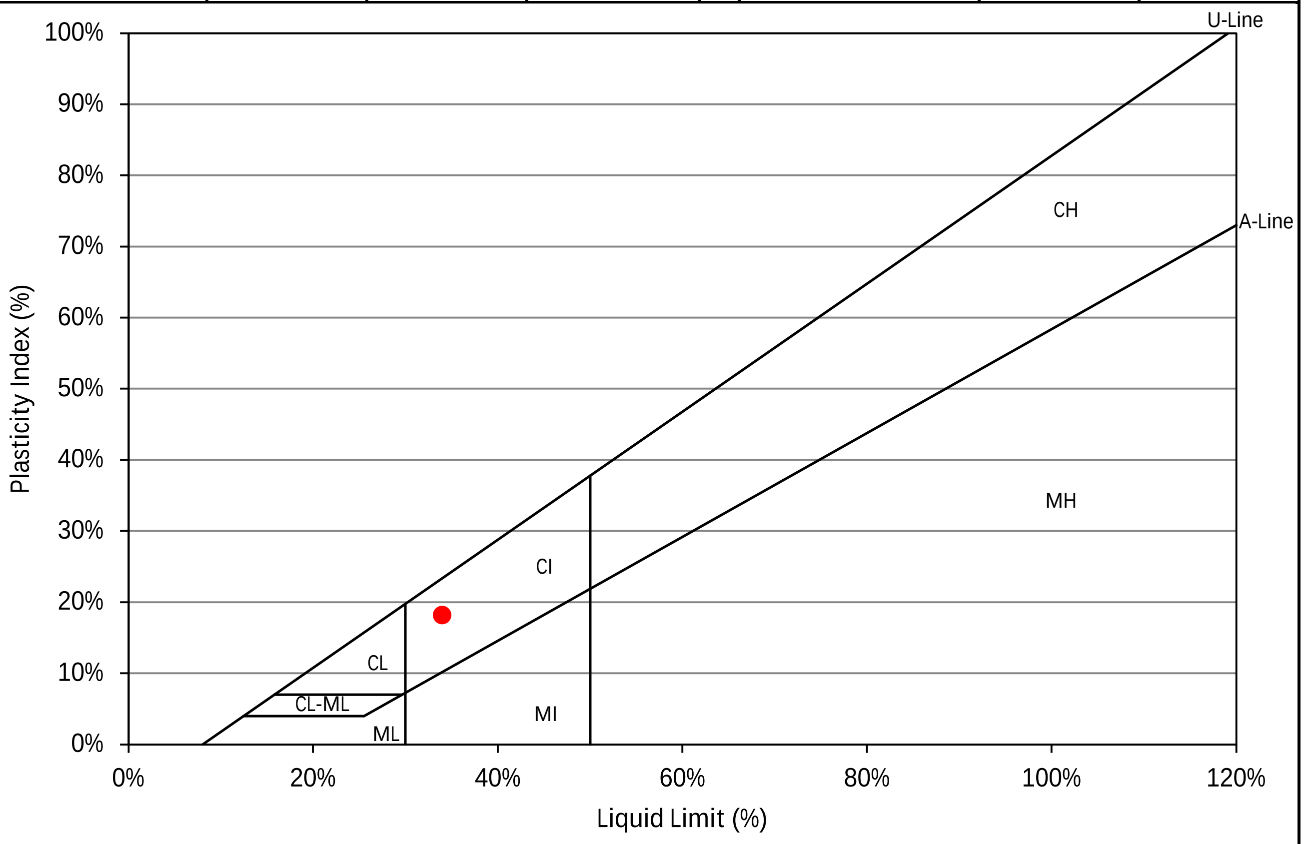
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-01	Sample Date:	1/25-26/2021
Sample Depth:	2.29 - 2.44 m	Lab Technician:	EManimbao
Sample Number:	G3	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	29	20	18
Wet Sample (g)	9.1	10.1	8.6
Dry Sample (g)	6.8	7.5	6.4
Water Content (%)	33.3%	34.8%	35.2%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.3	6.2
Dry Sample (g)	5.4	5.3
Water Content (%)	16.1%	15.5%



Liquid Limit (%): 34.0%	Plastic Limit (%): 15.8%	Plasticity Index (%): 18.2%

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

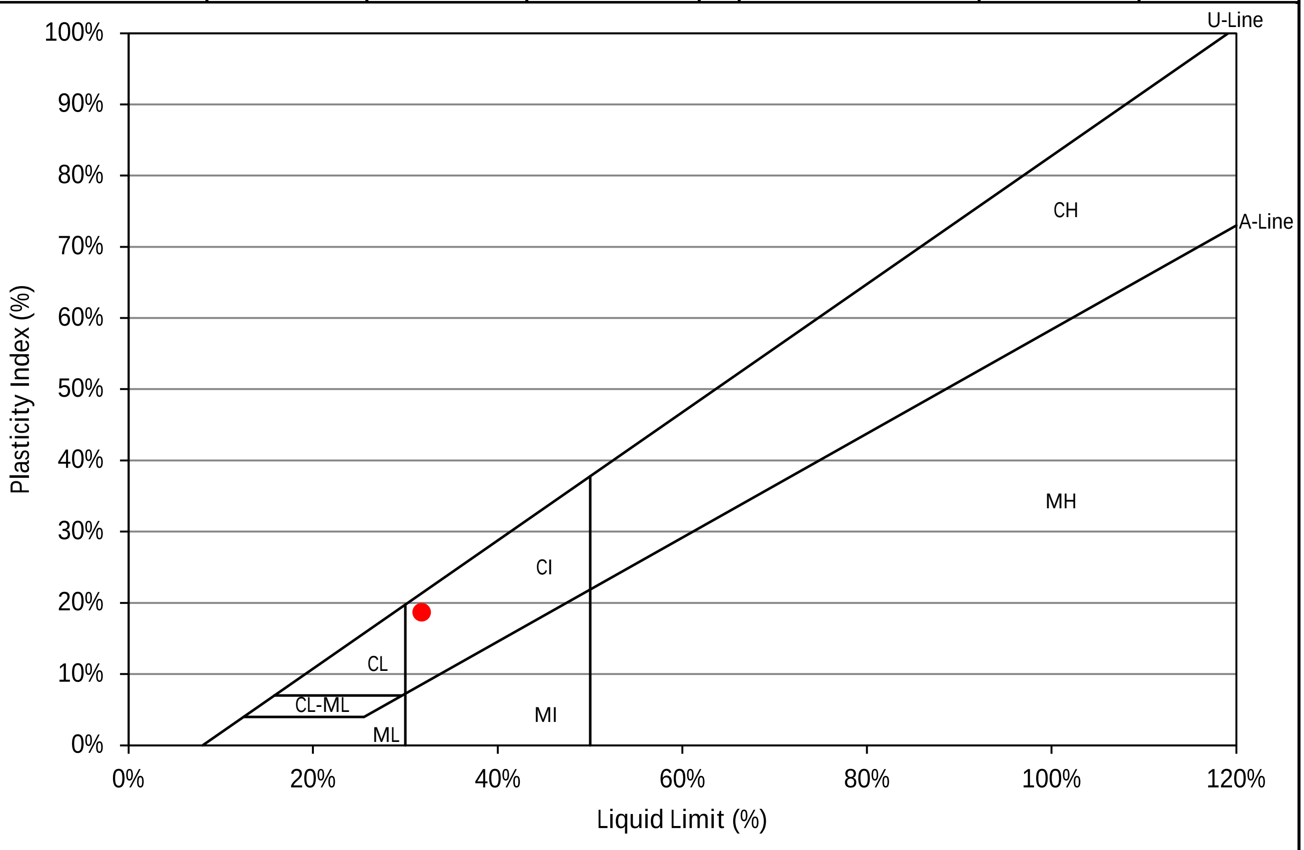
Project Name:	HRRC Phase 3		Supplier:	AECOM
Project Number:	60645745		Specification:	N/A
Client:	City of Winnipeg		Field Technician:	RHarras
Sample Location:	TH21-01		Sample Date:	1/25-26/2021
Sample Depth:	3.81 - 3.96 m		Lab Technician:	EManimbao
Sample Number:	G5		Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	25	17
Wet Sample (g)	8.4	11.0	9.2
Dry Sample (g)	6.4	8.4	6.9
Water Content (%)	30.4%	31.7%	33.0%

Plastic Limit		
Trial	1	2
Wet Sample (g)	7.2	6.9
Dry Sample (g)	6.4	6.1
Water Content (%)	13.0%	13.2%



Liquid Limit (%): 31.8%	Plastic Limit (%): 13.1%	Plasticity Index (%): 18.7%

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

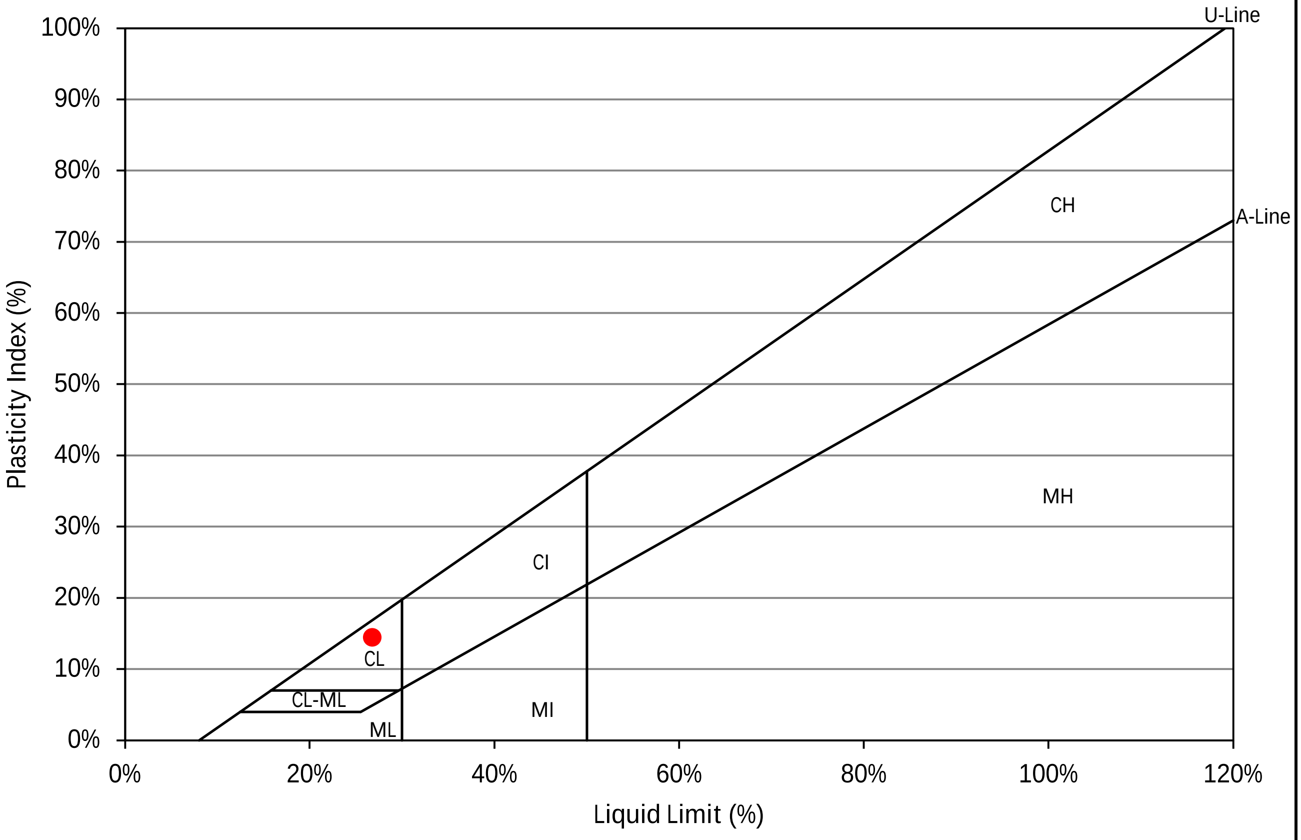
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-01	Sample Date:	1/25-26/2021
Sample Depth:	5.33 - 5.49 m	Lab Technician:	EManimbao
Sample Number:	G7	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	35	26	21
Wet Sample (g)	10.5	11.4	11.7
Dry Sample (g)	8.4	9.0	9.2
Water Content (%)	25.6%	26.7%	27.5%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.7	6.8
Dry Sample (g)	6.0	6.0
Water Content (%)	12.4%	12.2%



Liquid Limit (%): 26.8%	Plastic Limit (%): 12.3%	Plasticity Index (%): 14.4%

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R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

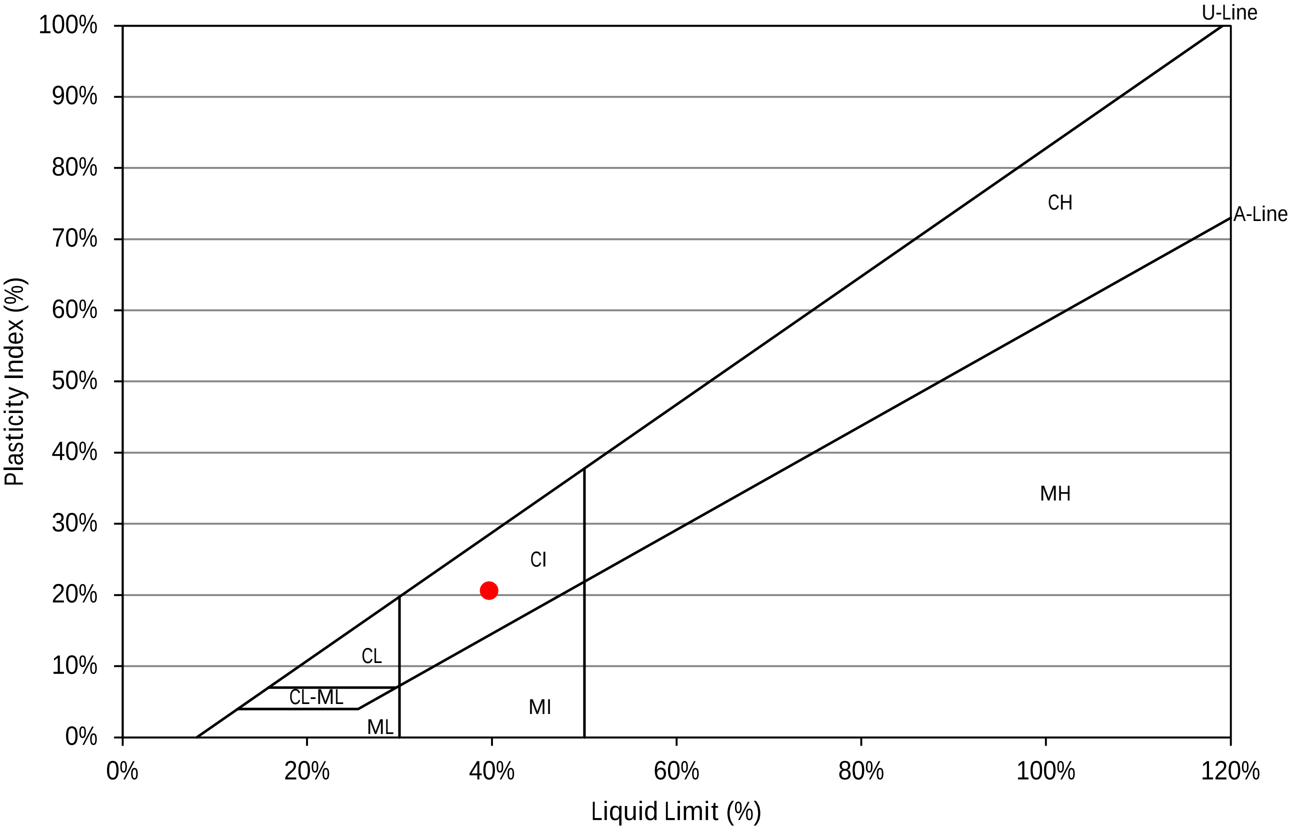
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-02	Sample Date:	1/25-26/2021
Sample Depth:	2.29 - 2.44 m	Lab Technician:	EManimbao
Sample Number:	G3	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	32	26	21
Wet Sample (g)	9.4	10.7	10.7
Dry Sample (g)	6.8	7.6	7.6
Water Content (%)	39.0%	39.5%	40.1%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.1	6.4
Dry Sample (g)	5.1	5.4
Water Content (%)	19.2%	19.0%



Liquid Limit (%): 39.7% | Plastic Limit (%): 19.1% | Plasticity Index (%): 20.6%

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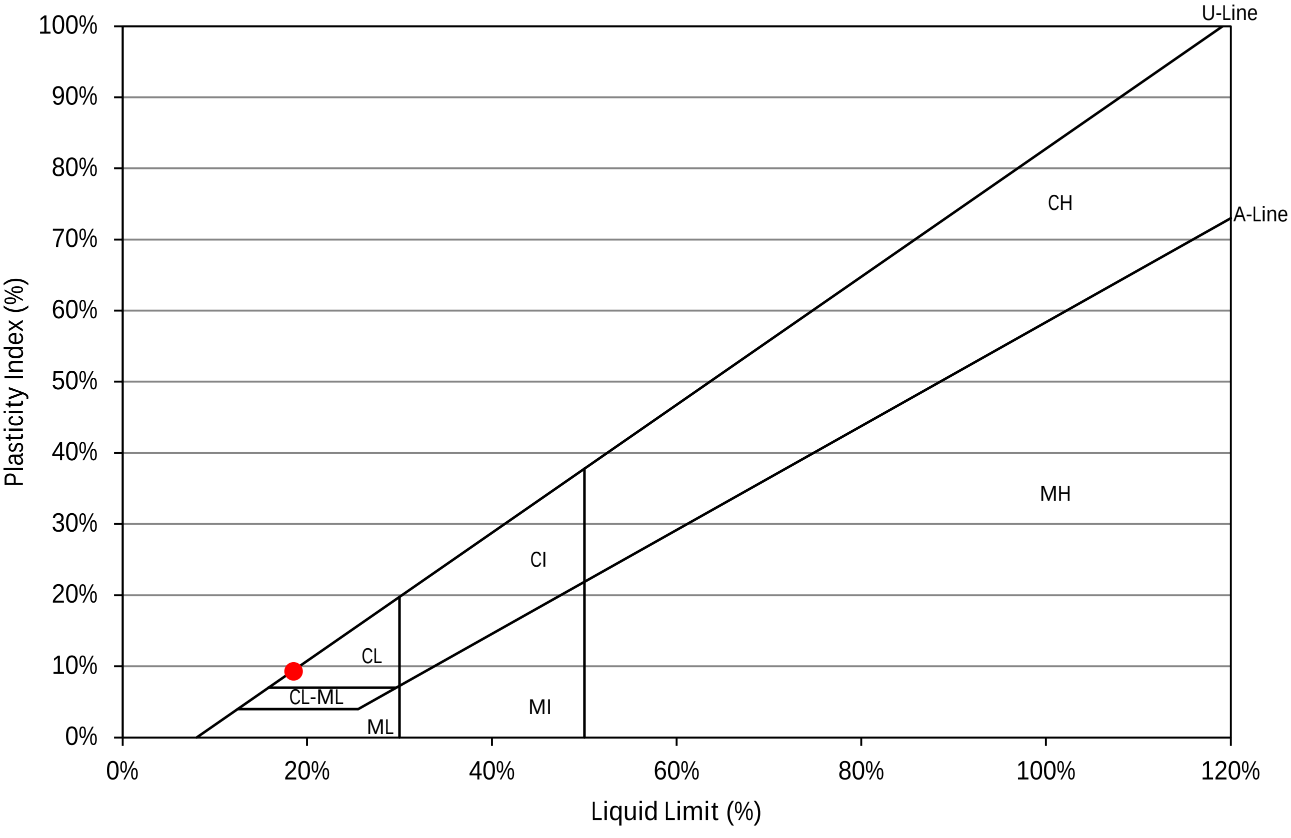
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-02	Sample Date:	1/25-26/2021
Sample Depth:	3.81 - 3.96 m	Lab Technician:	EManimbao
Sample Number:	G5	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	25	16
Wet Sample (g)	11.9	11.4	13.0
Dry Sample (g)	10.1	9.7	10.9
Water Content (%)	17.7%	18.4%	19.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.1	6.4
Dry Sample (g)	5.6	5.8
Water Content (%)	9.2%	9.3%



Liquid Limit (%): 18.5%	Plastic Limit (%): 9.2%	Plasticity Index (%): 9.3%

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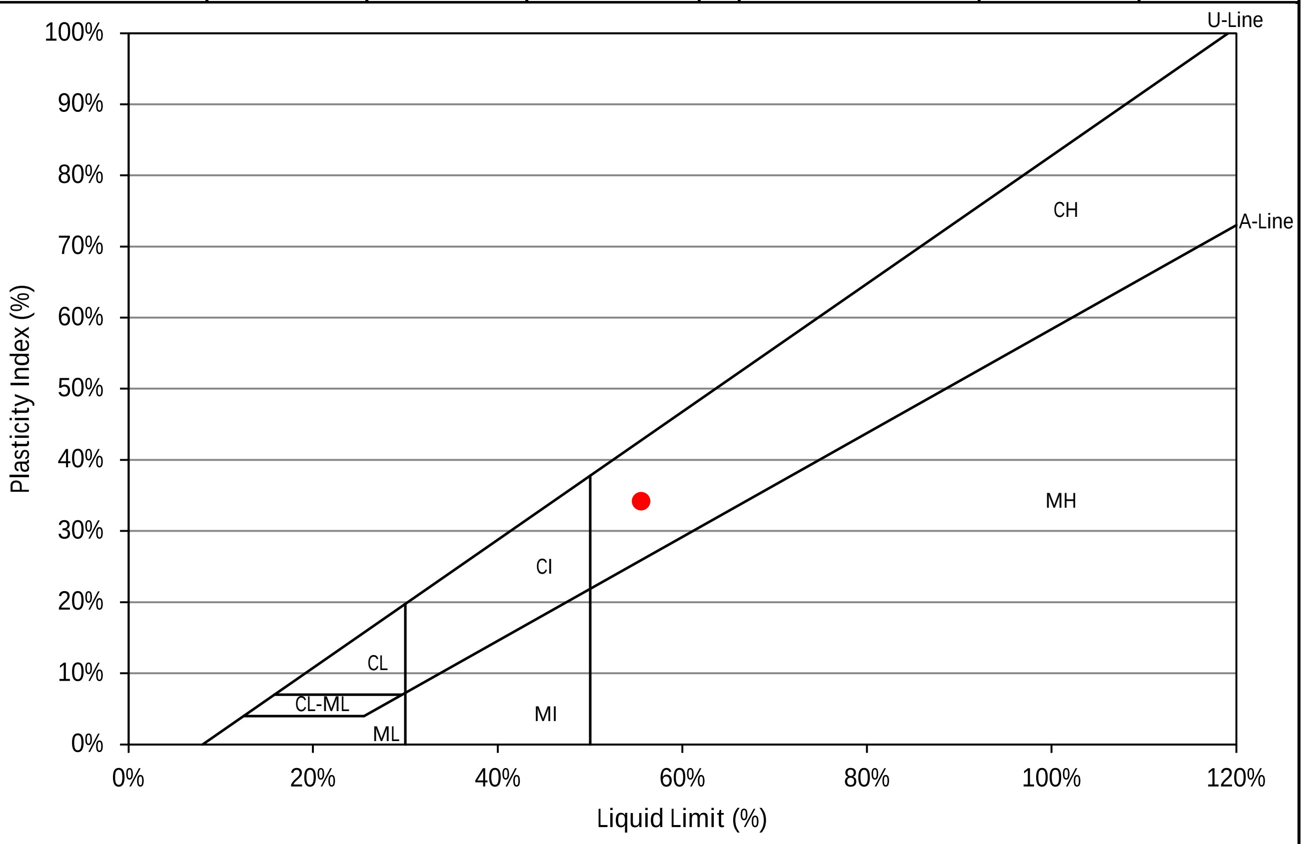
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-03	Sample Date:	1/25-26/2021
Sample Depth:	0.76 - 0.91 m	Lab Technician:	EManimbao
Sample Number:	G1	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	27	21	17
Wet Sample (g)	8.6	8.7	8.4
Dry Sample (g)	5.6	5.6	5.3
Water Content (%)	55.1%	56.5%	57.8%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.2	5.9
Dry Sample (g)	5.1	4.9
Water Content (%)	21.4%	21.3%



Liquid Limit (%): 55.5%	Plastic Limit (%): 21.3%	Plasticity Index (%): 34.2%

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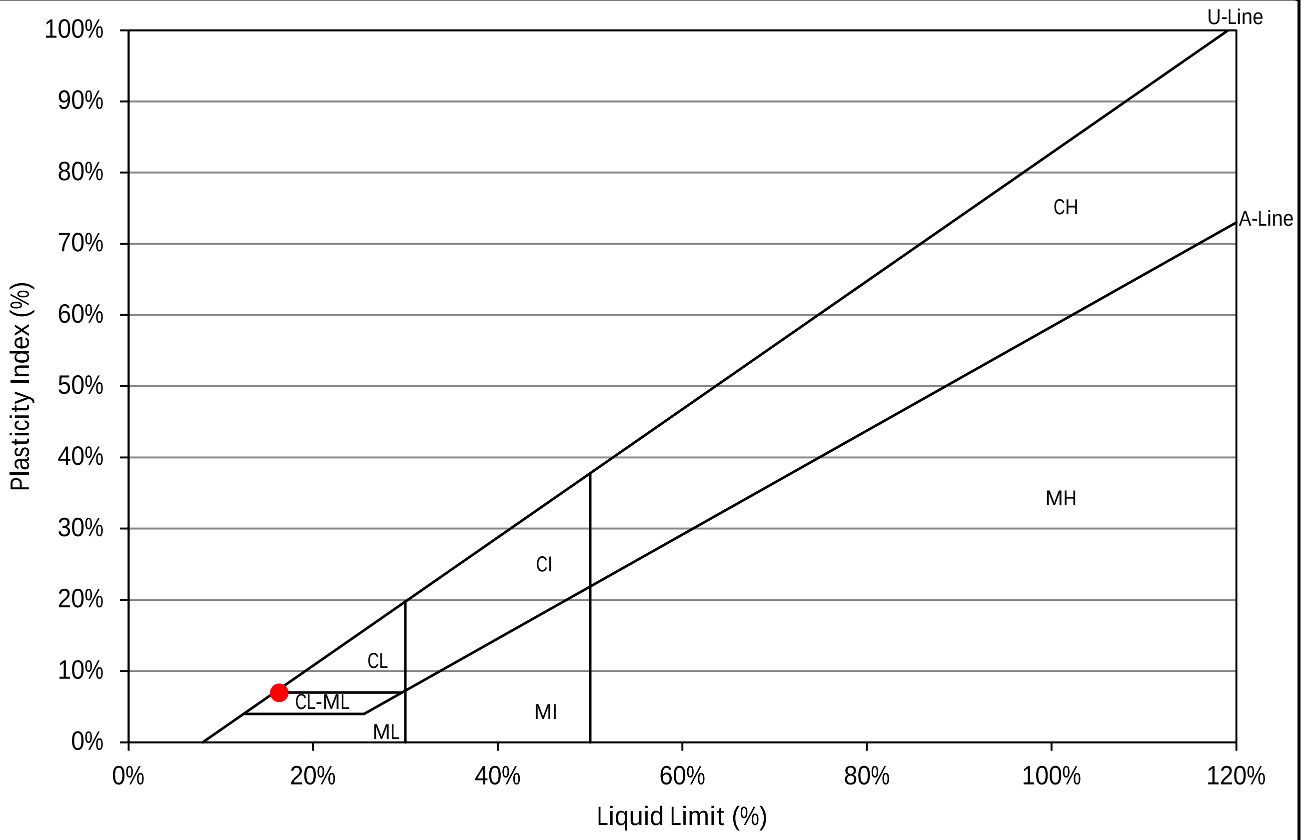
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-03	Sample Date:	1/25-26/2021
Sample Depth:	2.29 - 2.44 m	Lab Technician:	EManimbao
Sample Number:	G3	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	32	21	15
Wet Sample (g)	10.9	12.1	11.1
Dry Sample (g)	9.5	10.4	9.4
Water Content (%)	15.4%	16.9%	18.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.6	6.2
Dry Sample (g)	6.0	5.7
Water Content (%)	9.3%	9.5%



Liquid Limit (%): 16.3%	Plastic Limit (%): 9.4%	Plasticity Index (%): 7.0%

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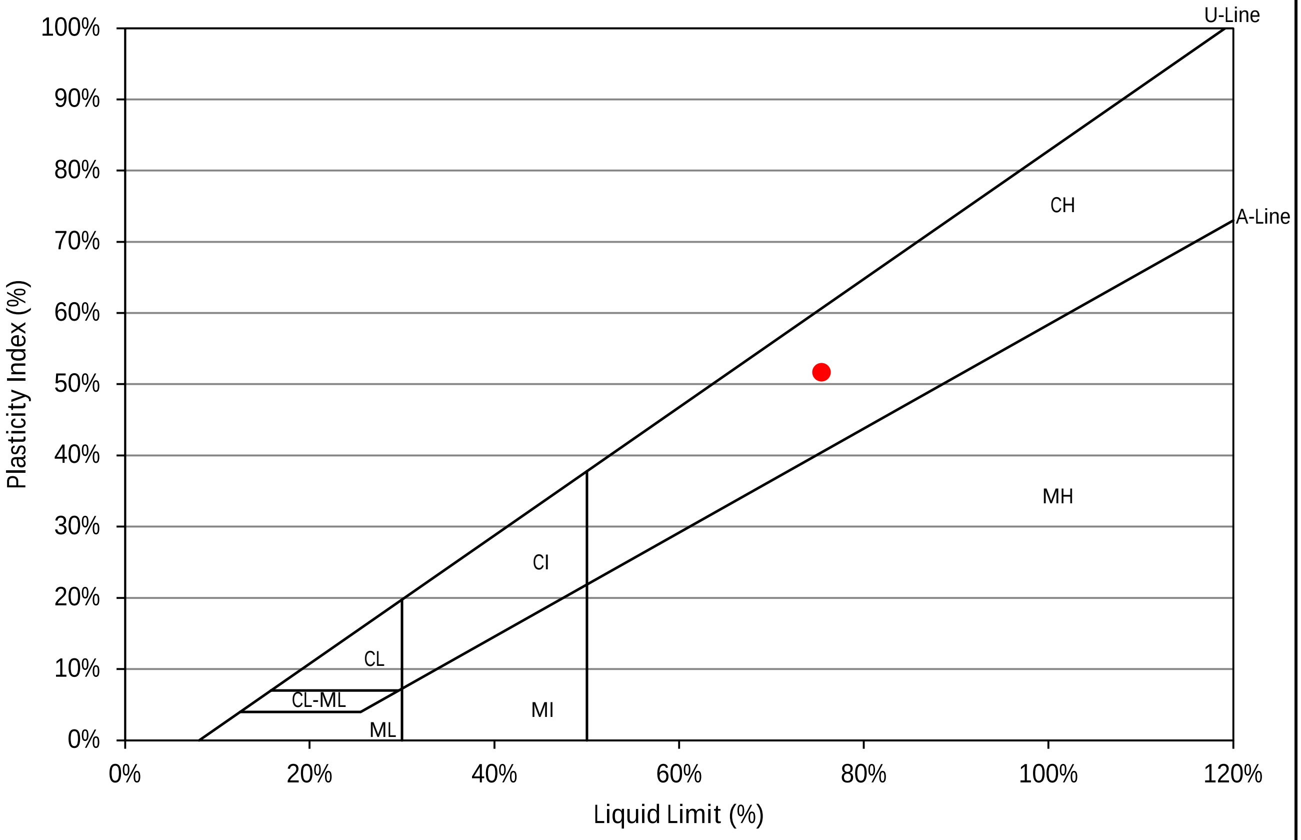
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-04	Sample Date:	1/25-26/2021
Sample Depth:	0.76 - 0.91 m	Lab Technician:	EManimbao
Sample Number:	G1	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	33	27	18
Wet Sample (g)	8.9	8.6	7.9
Dry Sample (g)	5.2	4.9	4.4
Water Content (%)	72.7%	74.8%	78.6%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.0	6.3
Dry Sample (g)	4.9	5.1
Water Content (%)	23.6%	23.9%



Liquid Limit (%): 75.4%	Plastic Limit (%): 23.8%	Plasticity Index (%): 51.7%

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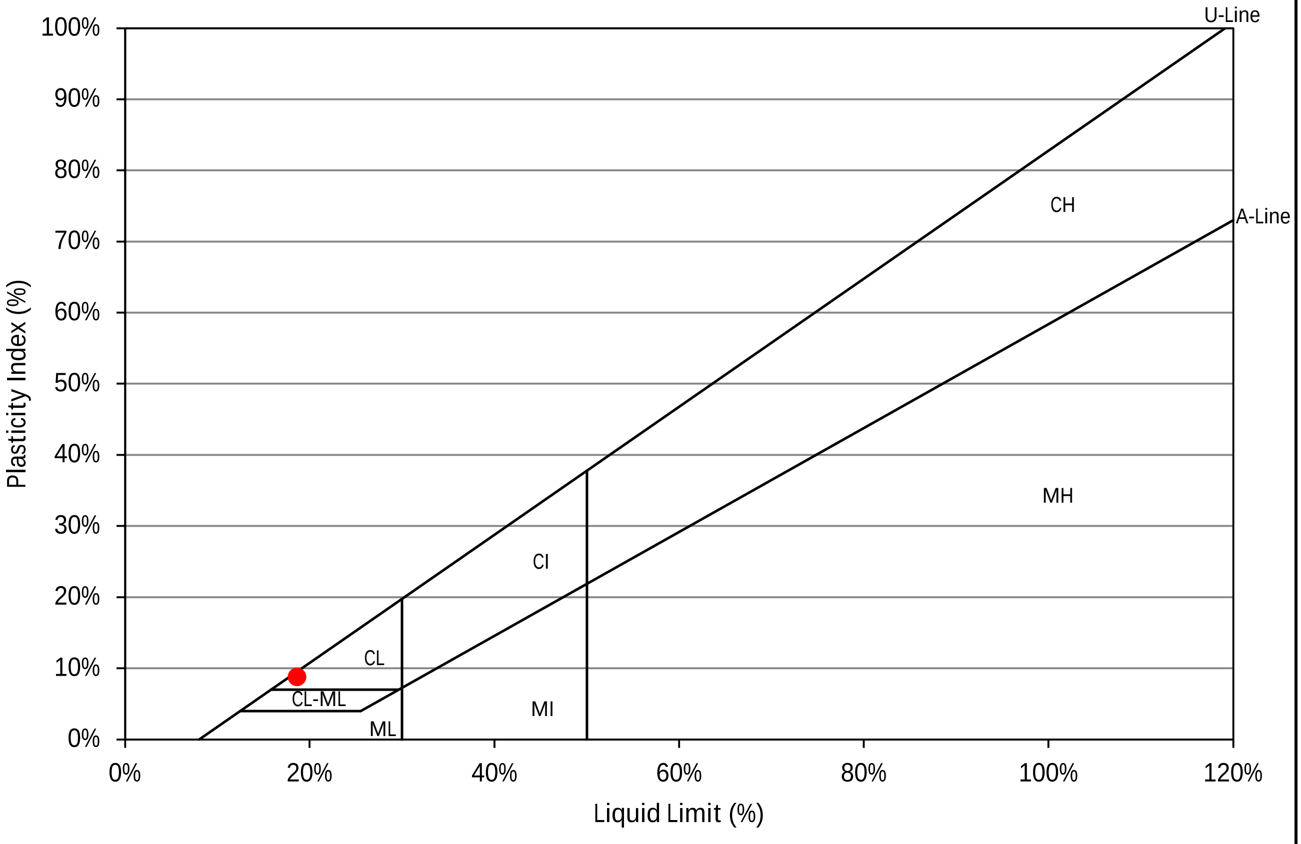
Project Name:	HRRC Phase 3	Supplier:	AECOM
Project Number:	60645745	Specification:	N/A
Client:	City of Winnipeg	Field Technician:	RHarras
Sample Location:	TH21-04	Sample Date:	1/25-26/2021
Sample Depth:	2.29 - 2.44 m	Lab Technician:	EManimbao
Sample Number:	G3	Date Tested:	February 16, 2021

Atterberg Limits (ASTM D4318)

Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Liquid Limit			
Blows	34	22	15
Wet Sample (g)	9.5	12.0	11.8
Dry Sample (g)	8.0	10.1	9.9
Water Content (%)	18.0%	18.7%	19.4%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.2	6.2
Dry Sample (g)	5.7	5.7
Water Content (%)	9.6%	10.1%



Liquid Limit (%): 18.6%	Plastic Limit (%): 9.9%	Plasticity Index (%): 8.8%

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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MATERIALS LABORATORY
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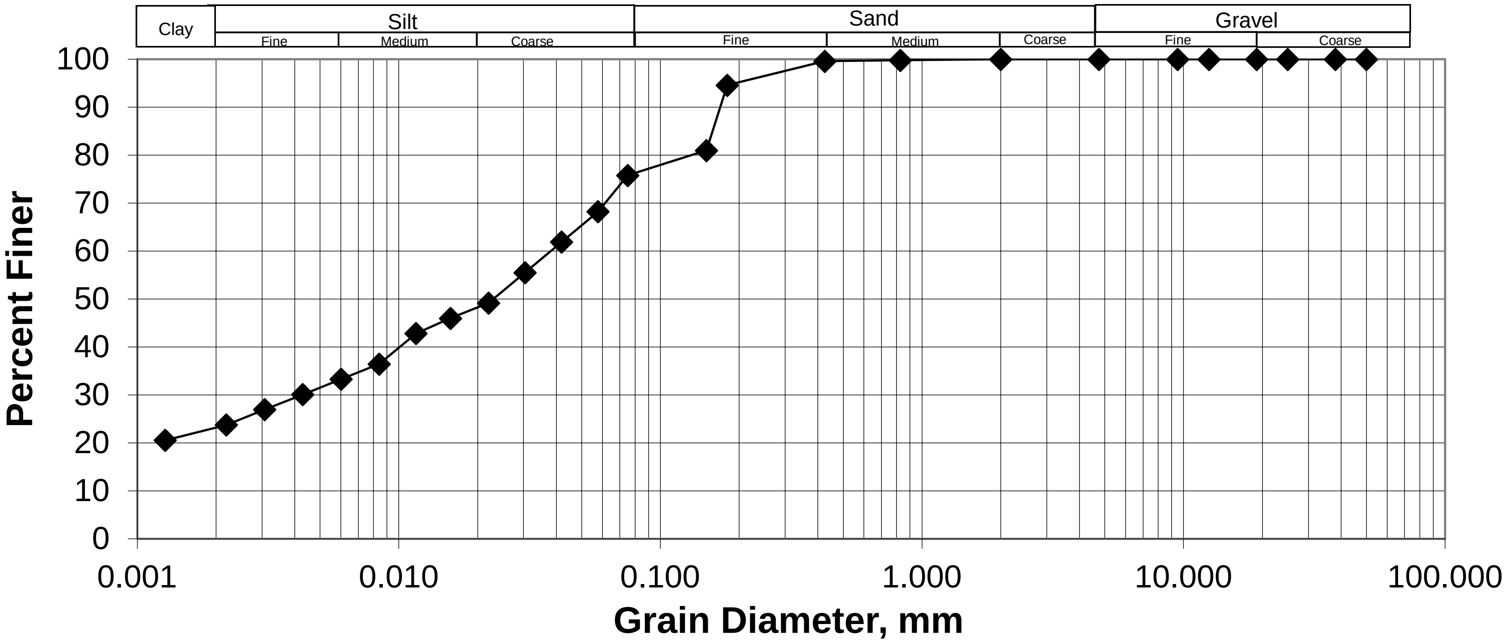
Job No.:	60645745	Hole No.:	TH21-01
Client:	City of Winnipeg	Sample No.:	G3
Project :	HRRC Phase 3	Depth:	2.29 - 2.44 m
Date Tested:	11-Feb-21	Date Sampled:	Varies
Tested By:	EManimbao	Sampled By:	AECOM

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	75.8
38.0	100.0	2.00	100.0	0.0577	68.2
25.0	100.0	0.825	99.8	0.0419	61.9
19.0	100.0	0.425	99.6	0.0304	55.5
12.5	100.0	0.18	94.6	0.0220	49.2
9.5	100.0	0.15	81.0	0.0157	46.0
4.75	100.0	0.075	75.8	0.0116	42.8
				0.0084	36.5
				0.0060	33.3
				0.0043	30.1
				0.0031	26.9
				0.0022	23.8
				0.0013	20.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**52.7%**
Sand	**24.2%**	**Clay**	**23.1%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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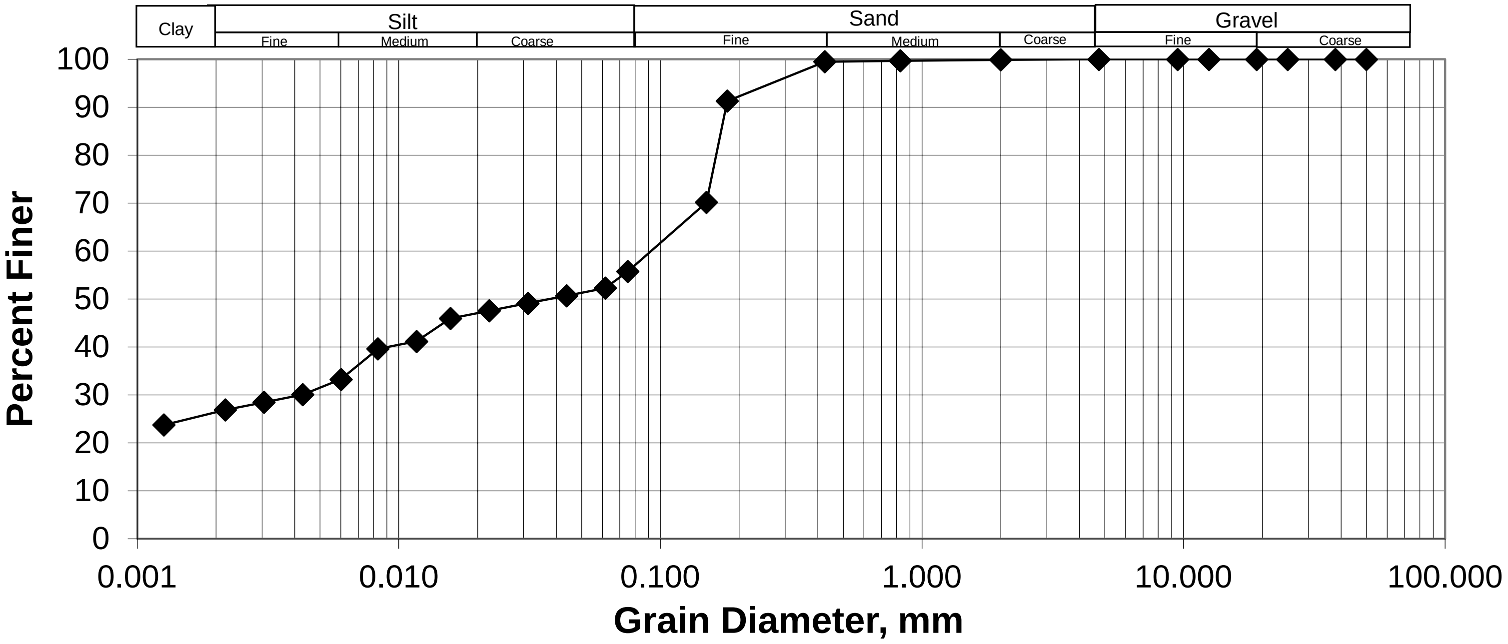
Job No.:	60645745	Hole No.:	TH21-01
Client:	City of Winnipeg	Sample No.:	G5
Project :	HRRC Phase 3	Depth:	3.81 - 3.96 m
Date Tested:	11-Feb-21	Date Sampled:	Varies
Tested By:	EManimbao	Sampled By:	AECOM

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	55.8
38.0	100.0	2.00	99.9	0.0615	52.3
25.0	100.0	0.825	99.7	0.0437	50.7
19.0	100.0	0.425	99.5	0.0311	49.1
12.5	100.0	0.18	91.3	0.0221	47.5
9.5	100.0	0.15	70.1	0.0157	46.0
4.75	100.0	0.075	55.8	0.0117	41.2
				0.0083	39.6
				0.0060	33.3
				0.0043	30.1
				0.0030	28.5
				0.0022	26.9
				0.0013	23.7

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**29.6%**
Sand	**44.2%**	**Clay**	**26.2%**

GRAIN SIZE DISTRIBUTION

(ASTM D422-63)

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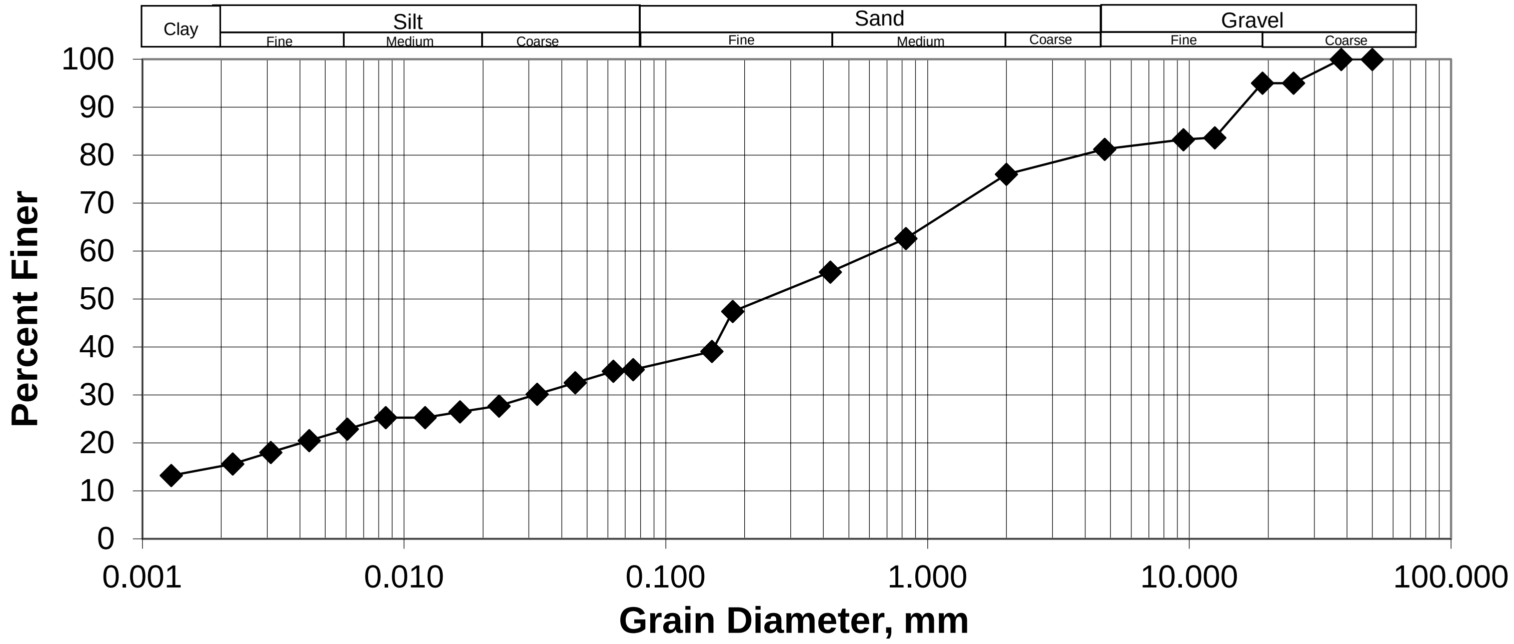
Job No.: 60645745
Client: City of Winnipeg
Project : HRRC Phase 3
Date Tested: 11-Feb-21
Tested By: EManimbao

Hole No.: TH21-01
Sample No.: G7
Depth: 5.33 - 2.44 m
Date Sampled: Varies
Sampled By: AECOM

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	81.3	0.0750	35.3
38.0	100.0	2.00	76.0	0.0629	35.0
25.0	95.0	0.825	62.7	0.0450	32.6
19.0	95.0	0.425	55.7	0.0322	30.1
12.5	83.7	0.18	47.4	0.0230	27.7
9.5	83.3	0.15	39.1	0.0164	26.5
4.75	81.3	0.075	35.3	0.0120	25.3
				0.0085	25.3
				0.0061	22.9
				0.0043	20.5
				0.0031	18.1
				0.0022	15.7
				0.0013	13.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**18.7%**	**Silt**	**20.2%**
Sand	**46.0%**	**Clay**	**15.1%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

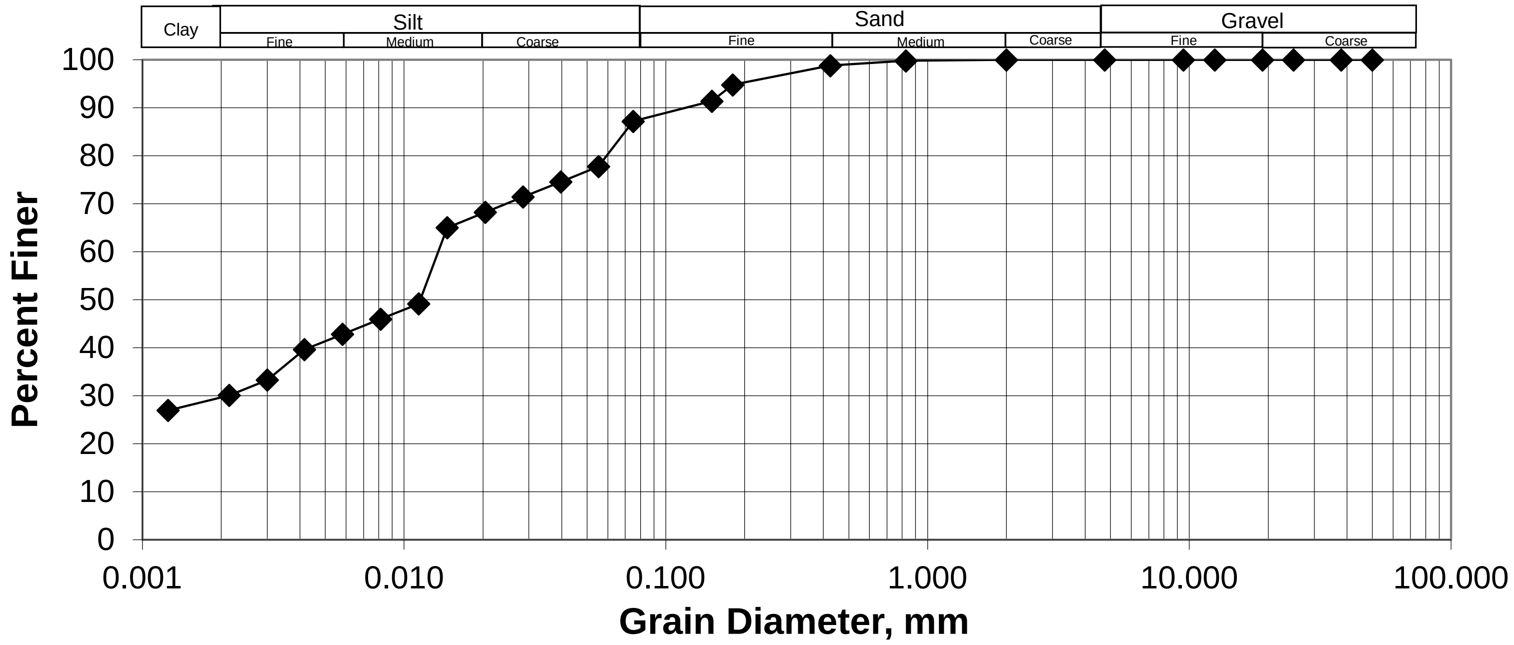
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381   **fax** (204) 284-2040

Job No.: 60645745
Client: City of Winnipeg
Project : HRRC Phase 3
Date Tested: 11-Feb-21
Tested By: EManimbao

Hole No.: TH21-02
Sample No.: G3
Depth: 2.29 - 2.44 m
Date Sampled: Varies
Sampled By: AECOM

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	87.2
38.0	100.0	2.00	100.0	0.0552	77.8
25.0	100.0	0.825	99.8	0.0396	74.6
19.0	100.0	0.425	98.8	0.0284	71.4
12.5	100.0	0.18	94.8	0.0204	68.2
9.5	100.0	0.15	91.4	0.0146	65.1
4.75	100.0	0.075	87.2	0.0114	49.2
				0.0081	46.0
				0.0058	42.8
				0.0042	39.6
				0.0030	33.3
				0.0021	30.1
				0.0013	26.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**57.5%**
Sand	**12.8%**	**Clay**	**29.7%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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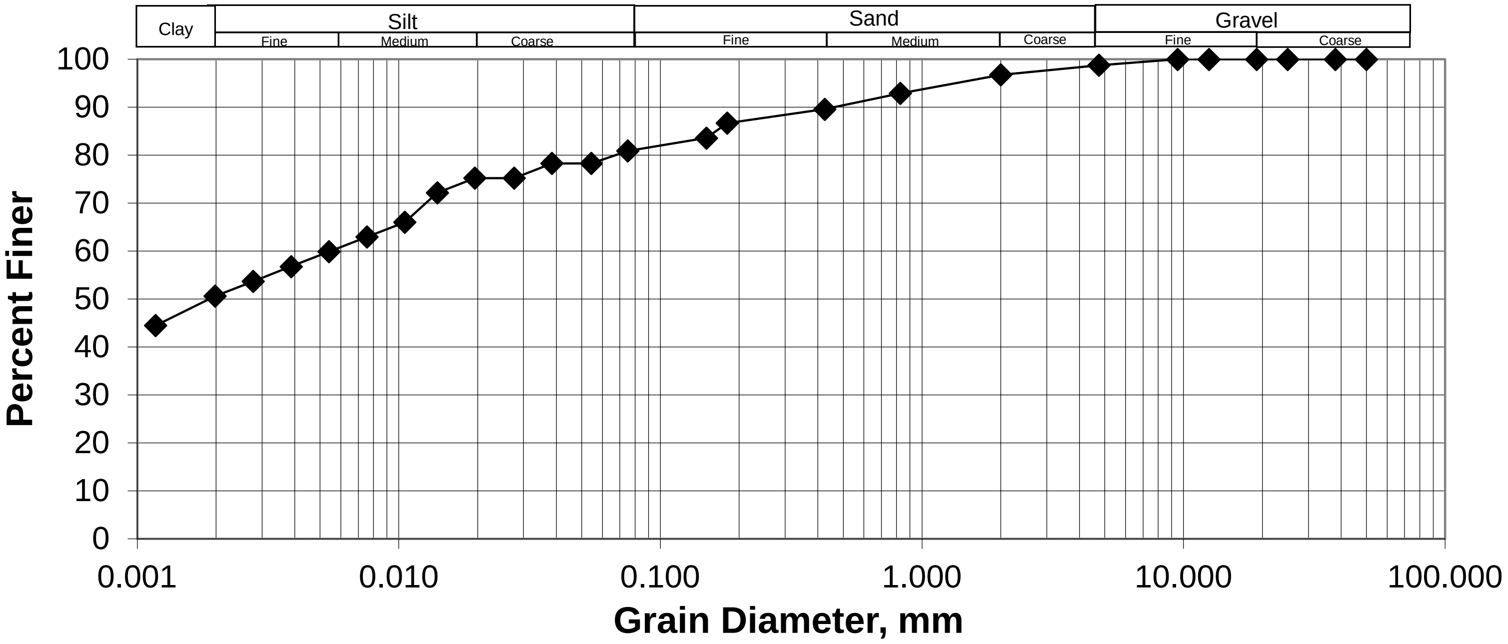
Job No.:	60645745	Hole No.:	TH21-03
Client:	City of Winnipeg	Sample No.:	G1
Project :	HRRC Phase 3	Depth:	0.76 - 0.91 m
Date Tested:	11-Feb-21	Date Sampled:	Varies
Tested By:	EManimbao	Sampled By:	AECOM

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	98.7	0.0750	80.9
38.0	100.0	2.00	96.8	0.0544	78.3
25.0	100.0	0.825	92.9	0.0385	78.3
19.0	100.0	0.425	89.6	0.0276	75.2
12.5	100.0	0.18	86.7	0.0195	75.2
9.5	100.0	0.15	83.6	0.0140	72.2
4.75	98.7	0.075	80.9	0.0105	66.0
				0.0075	62.9
				0.0054	59.9
				0.0039	56.8
				0.0028	53.7
				0.0020	50.6
				0.0012	44.5

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**1.3%**	**Silt**	**30.3%**
Sand	**17.8%**	**Clay**	**50.6%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

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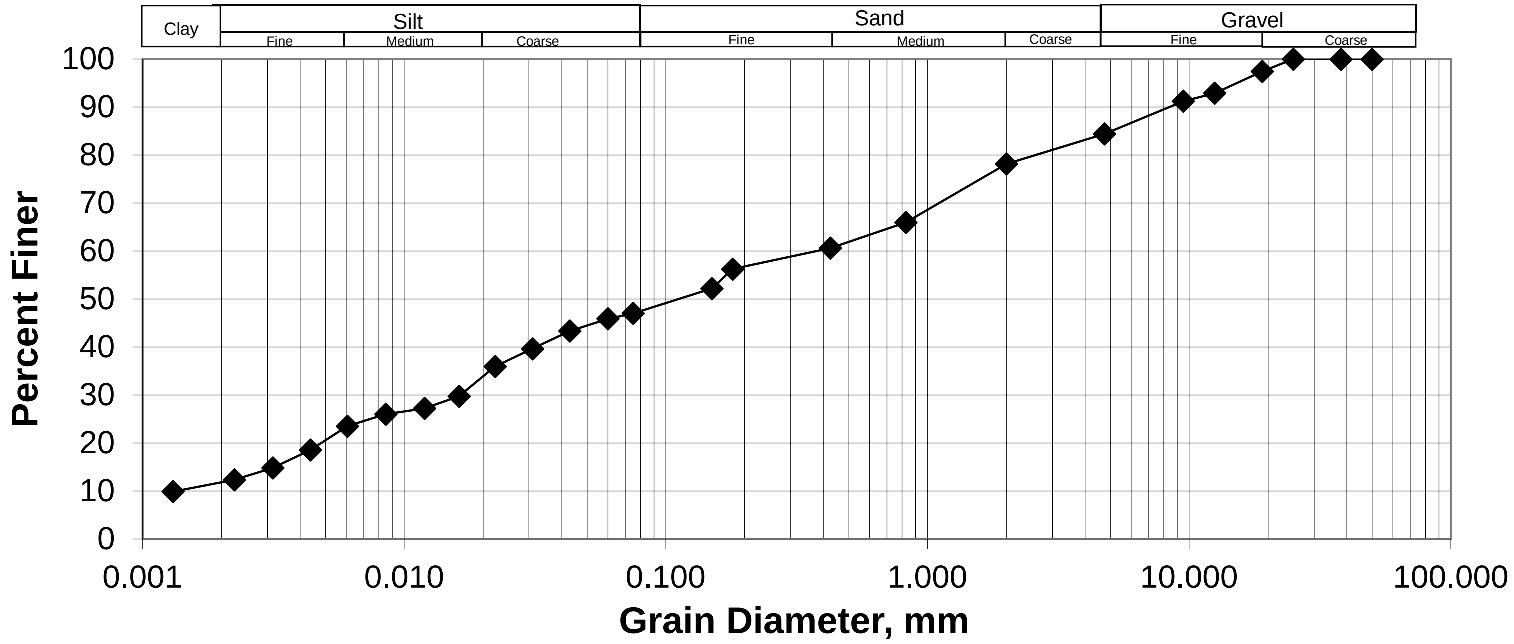
Job No.:	60645745	Hole No.:	TH21-03
Client:	City of Winnipeg	Sample No.:	G3
Project :	HRRC Phase 3	Depth:	2.29 - 2.44 m
Date Tested:	11-Feb-21	Date Sampled:	Varies
Tested By:	EManimbao	Sampled By:	AECOM

GRAVEL SIZES		SAND SIZES		FINES	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	84.4	0.0750	47.0
38.0	100.0	2.00	78.1	0.0600	45.9
25.0	100.0	0.825	65.9	0.0429	43.4
19.0	97.4	0.425	60.6	0.0309	39.7
12.5	92.9	0.18	56.3	0.0223	35.9
9.5	91.2	0.15	52.2	0.0162	29.7
4.75	84.4	0.075	47.0	0.0119	27.3
				0.0085	26.0
				0.0061	23.5
				0.0044	18.6
				0.0031	14.8
				0.0022	12.4
				0.0013	9.9

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**15.6%**	**Silt**	**35.2%**
Sand	**37.4%**	**Clay**	**11.8%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

MATERIALS LABORATORY
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99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
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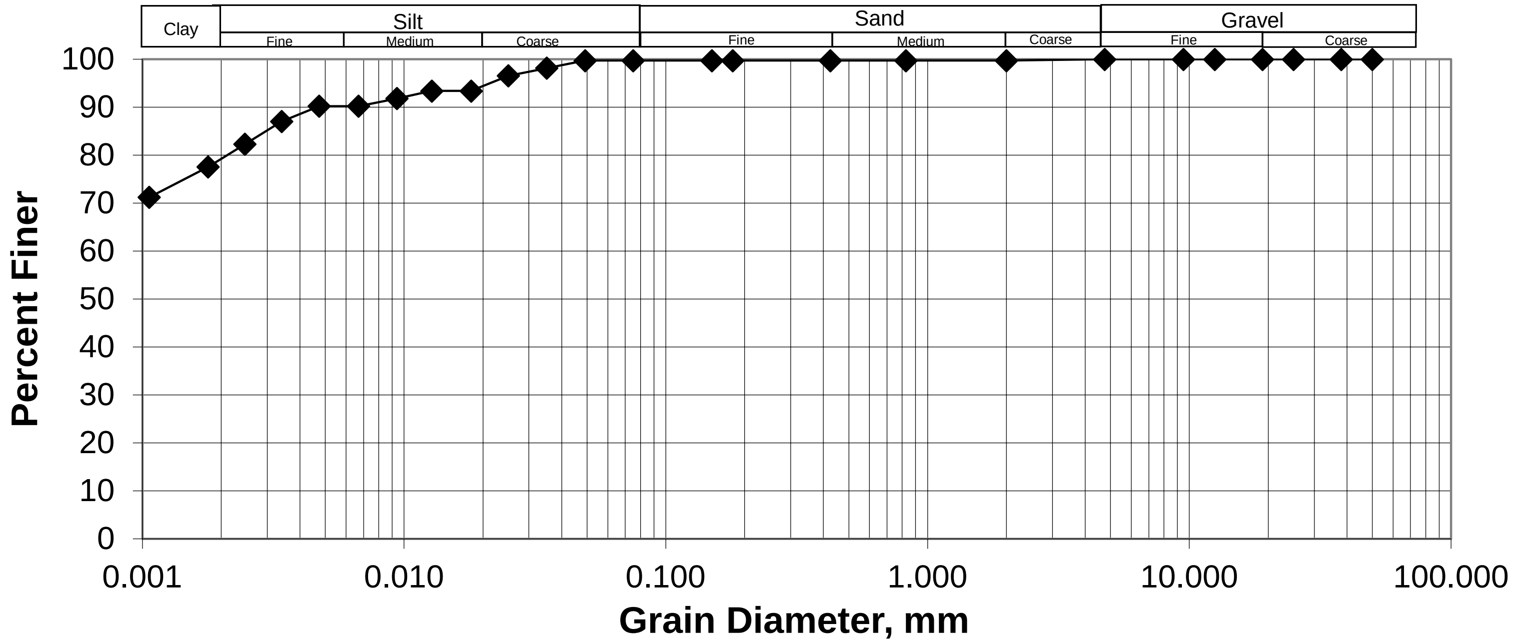
Job No.:	60645745	Hole No.:	TH21-04
Client:	City of Winnipeg	Sample No.:	G1
Project :	HRRC Phase 3	Depth:	0.76 - 0.91 m
Date Tested:	11-Feb-21	Date Sampled:	Varies
Tested By:	EManimbao	Sampled By:	AECOM

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	100.0	0.0750	99.7
38.0	100.0	2.00	99.7	0.0491	99.7
25.0	100.0	0.825	99.7	0.0351	98.1
19.0	100.0	0.425	99.7	0.0250	96.6
12.5	100.0	0.18	99.7	0.0180	93.4
9.5	100.0	0.15	99.7	0.0127	93.4
4.75	100.0	0.075	99.7	0.0094	91.8
				0.0067	90.2
				0.0047	90.2
				0.0034	87.1
				0.0025	82.3
				0.0018	77.5
				0.0011	71.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**20.8%**
Sand	**0.3%**	**Clay**	**78.9%**

GRAIN SIZE DISTRIBUTION
(ASTM D422-63)

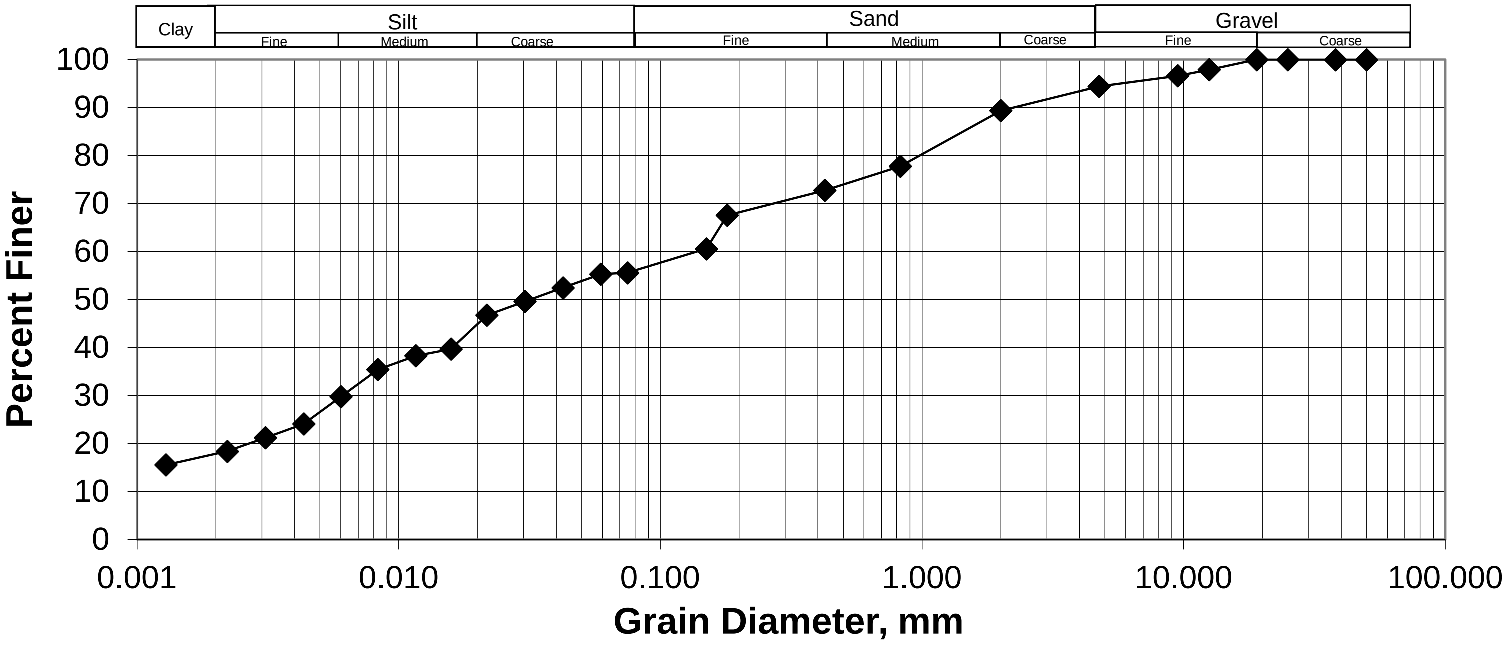
AECOM
MATERIALS LABORATORY
AECOM
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381   **fax** (204) 284-2040

Job No.: 60645745
Client: City of Winnipeg
Project : HRRC Phase 3
Date Tested: 11-Feb-21
Tested By: EManimbao

Hole No.: TH21-04
Sample No.: G3
Depth: 2.29 - 2.44 m
Date Sampled: Varies
Sampled By: AECOM

GRAVEL SIZES		**SAND SIZES**		**FINES**	
Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing	Grain Size (mm.)	Total Percent Passing
50.0	100.0	4.75	94.4	0.0750	55.6
38.0	100.0	2.00	89.4	0.0592	55.3
25.0	100.0	0.825	77.8	0.0424	52.5
19.0	100.0	0.425	72.8	0.0304	49.6
12.5	97.9	0.18	67.6	0.0217	46.8
9.5	96.6	0.15	60.6	0.0158	39.7
4.75	94.4	0.075	55.6	0.0116	38.3
				0.0083	35.4
				0.0060	29.8
				0.0043	24.1
				0.0031	21.2
				0.0022	18.4
				0.0013	15.6

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**5.6%**	**Silt**	**37.8%**
Sand	**38.8%**	**Clay**	**17.8%**

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: City of Winnipeg
PROJECT: HRRC Phase 3
JOB NO.: 60645745

TEST HOLE NO.:	**TH21-01**
SAMPLE NO.:	**T4B**
SAMPLE DEPTH:	3.05 - 3.66 m
DATE TESTED:	2-Feb-21
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.35
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**34.3**
Undrained Shear Strength (ksf)	0.72
POCKET PENETROMETER	
Reading - Qu (tsf)	0.75
Undrained Shear Strength (kPa)	**35.9**
Reading - Qu (tsf)	0.75
Undrained Shear Strength (kPa)	**35.9**
Reading - Qu (tsf)	0.75
Undrained Shear Strength (kPa)	**35.9**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**43.9**
Unconfined compressive strength (ksf)	**0.9**
Undrained Shear Strength (kPa)	**22.0**
Undrained Shear Strength (ksf)	**0.459**
MOISTURE CONTENT	
Tare Number	SG27
Wt. Sample wet + tare (g)	505.4
Wt. Sample dry + tare (g)	406.6
Wt. Tare (g)	8.3
Moisture Content %	**24.8**
BULK DENSITY	
Sample Wt. (g)	1216.1
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.20
Diameter 3 (cm)	7.30
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.20
Length 2 (cm)	15.20
Length 3 (cm)	15.30
Avg. Length (cm)	**15.23**
Volume (cm^3)	626.0
Moisture content (%)	24.8
Bulk Density (g/cm^3)	1.943
Bulk Density (kN/m^3)	**19.1**
Bulk Density (pcf)	**121.3**
Dry Density (kN/m^3)	**15.27**

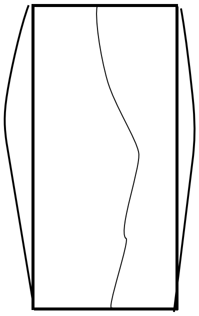
AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

AECOM

CLIENT:	City of Winnipeg
PROJECT:	HRRC Phase 3
JOB NO.:	60645745

TEST HOLE NO.:	TH21-01
SAMPLE NO.:	T4
SAMPLE DEPTH:	3.05 - 3.66 m
SAMPLE DATE:	
TEST DATE:	2-Feb-21

SOIL DESCRIPTION:	
CLAY - silty, trace to some sand, brown	
moist, firm, intermediate to high plasticity	
MOISTURE CONTENT:	24.8



FAILURE SKETCH

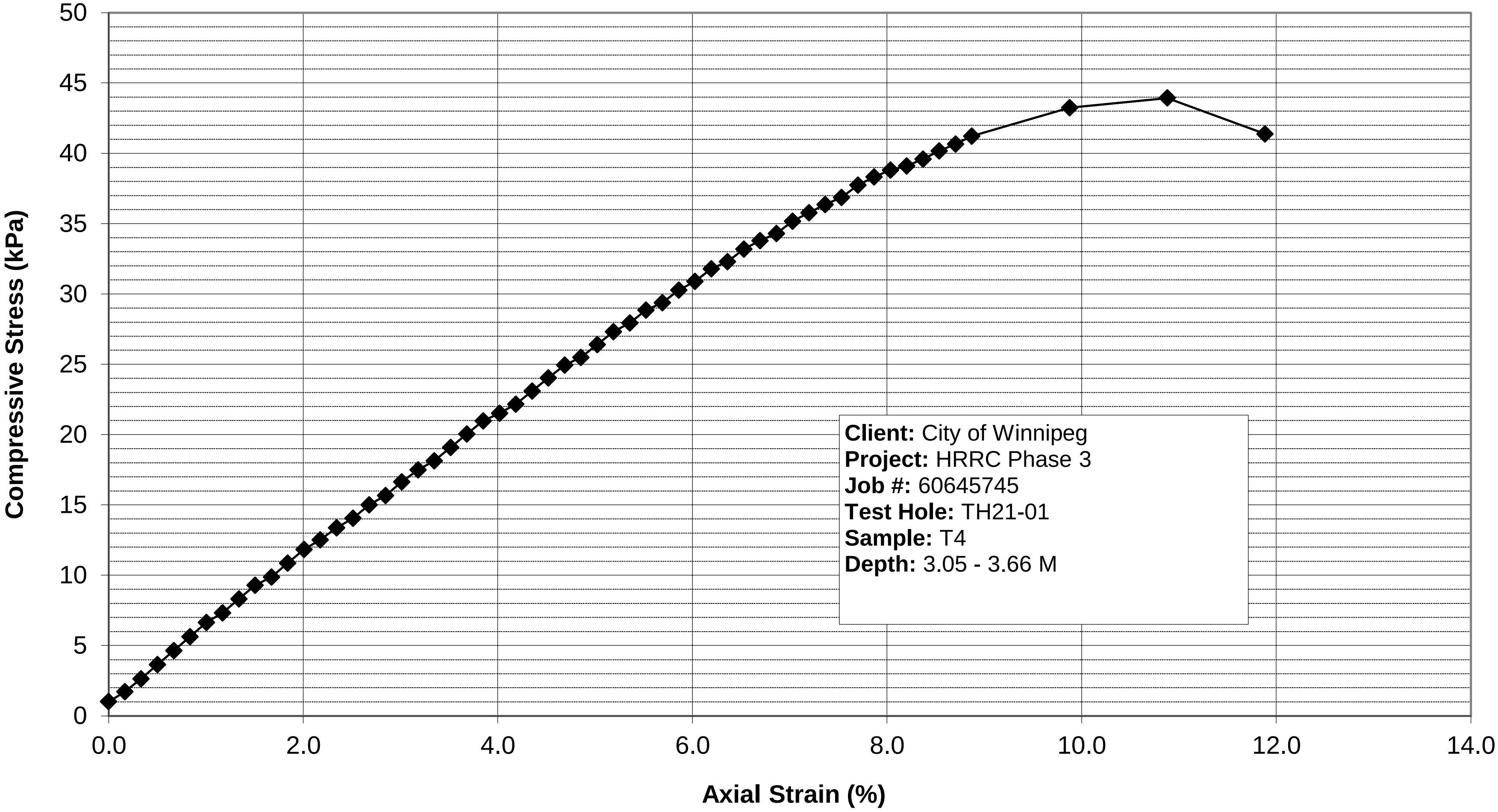
SAMPLE DIAM.(Do):	72.33	**(mm)**	**INITIAL AREA, Ao:**	4109.3	**(mm^2)**
SAMPLE LENGTH, (Lo):	152.33	**(mm)**	**PISTON RATE:**	0.0602	**(inches / minute)**
L / D RATIO:	2.11	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	1.00	**(0.5<R<2 % / minute)**

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	**PROVING RING**	**TOTAL AXIAL STRAIN, E_1**	**AVERAGE CROSS-SECTIONAL AREA, A**	**APPLIED AXIAL LOAD, P**	**COMPRESSIVE STRESS, σ_C**		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0001	0.00	6.37	0.94	0.15	0.021	1.0
0.02	0.0002	0.17	6.38	1.59	0.25	0.036	1.7
0.03	0.0003	0.33	6.39	2.44	0.38	0.055	2.6
0.04	0.0004	0.50	6.40	3.37	0.53	0.076	3.6
0.05	0.0005	0.67	6.41	4.31	0.67	0.097	4.6
0.06	0.0006	0.84	6.42	5.25	0.82	0.118	5.6
0.07	0.0007	1.00	6.43	6.18	0.96	0.138	6.6
0.08	0.0007	1.17	6.44	6.84	1.06	0.153	7.3
0.09	0.0008	1.34	6.46	7.78	1.20	0.173	8.3
0.10	0.0009	1.51	6.47	8.71	1.35	0.194	9.3
0.11	0.0010	1.67	6.48	9.28	1.43	0.206	9.9
0.12	0.0011	1.84	6.49	10.21	1.57	0.227	10.9
0.13	0.0012	2.01	6.50	11.15	1.72	0.247	11.8
0.14	0.0013	2.18	6.51	11.81	1.81	0.261	12.5
0.15	0.0014	2.34	6.52	12.65	1.94	0.279	13.4
0.16	0.0014	2.51	6.53	13.31	2.04	0.293	14.0
0.17	0.0015	2.68	6.54	14.24	2.18	0.313	15.0
0.18	0.0016	2.85	6.56	14.90	2.27	0.327	15.7
0.19	0.0017	3.01	6.57	15.84	2.41	0.347	16.6
0.20	0.0018	3.18	6.58	16.68	2.54	0.365	17.5
0.21	0.0019	3.35	6.59	17.33	2.63	0.379	18.1
0.22	0.0020	3.52	6.60	18.27	2.77	0.399	19.1
0.23	0.0021	3.68	6.61	19.21	2.90	0.418	20.0
0.24	0.0022	3.85	6.62	20.15	3.04	0.438	21.0
0.25	0.0022	4.02	6.64	20.71	3.12	0.449	21.5
0.26	0.0023	4.18	6.65	21.36	3.21	0.463	22.2
0.27	0.0024	4.35	6.66	22.30	3.35	0.482	23.1
0.28	0.0025	4.52	6.67	23.24	3.48	0.502	24.0
0.29	0.0026	4.69	6.68	24.17	3.62	0.521	24.9
0.30	0.0026	4.85	6.69	24.74	3.70	0.532	25.5
0.31	0.0027	5.02	6.71	25.67	3.83	0.551	26.4
0.32	0.0028	5.19	6.72	26.61	3.96	0.570	27.3
0.33	0.0029	5.36	6.73	27.27	4.05	0.583	27.9
0.34	0.0030	5.52	6.74	28.20	4.18	0.602	28.8
0.35	0.0031	5.69	6.75	28.77	4.26	0.613	29.4
0.36	0.0032	5.86	6.77	29.70	4.39	0.632	30.3
0.37	0.0032	6.03	6.78	30.36	4.48	0.645	30.9
0.38	0.0033	6.19	6.79	31.30	4.61	0.664	31.8
0.39	0.0034	6.36	6.80	31.86	4.68	0.674	32.3
0.40	0.0035	6.53	6.81	32.80	4.81	0.693	33.2
0.41	0.0036	6.70	6.83	33.45	4.90	0.706	33.8
0.42	0.0036	6.86	6.84	34.01	4.97	0.716	34.3
0.43	0.0037	7.03	6.85	34.95	5.10	0.735	35.2
0.44	0.0038	7.20	6.86	35.61	5.19	0.747	35.8
0.45	0.0039	7.37	6.88	36.26	5.27	0.759	36.4
0.46	0.0039	7.53	6.89	36.82	5.35	0.770	36.9
0.47	0.0040	7.70	6.90	37.76	5.47	0.788	37.7
0.48	0.0041	7.87	6.91	38.42	5.56	0.800	38.3
0.49	0.0042	8.03	6.93	38.98	5.63	0.810	38.8
0.50	0.0042	8.20	6.94	39.35	5.67	0.817	39.1
0.51	0.0043	8.37	6.95	39.92	5.74	0.827	39.6
0.52	0.0043	8.54	6.96	40.57	5.83	0.839	40.2
0.53	0.0044	8.70	6.98	41.13	5.90	0.849	40.7
0.54	0.0045	8.87	6.99	41.79	5.98	0.861	41.2
0.60	0.0047	9.88	7.07	44.32	6.27	0.903	43.2
0.66	0.0049	10.88	7.15	45.54	6.37	0.918	43.9
0.72	0.0046	11.89	7.23	43.38	6.00	0.864	41.4

UNCONFINED COMPRESSIVE STRENGTH, q_u:	43.93	kPa
(based on maximum q_u value)	0.918	ksf
UNDRAINED SHEAR STRENGTH, S_u:	21.97	kPa
(based on maximum q_u value)	0.459	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: City of Winnipeg
Project: HRRC Phase 3
Job #: 60645745
Test Hole: TH21-01
Sample: T4
Depth: 3.05 - 3.66 M

AECOM - SOILS LABORATORY
SHEAR STRENGTH, MOISTURE CONTENT & DENSITY CALCULATIONS

CLIENT: City of Winnipeg
PROJECT: HRRC Phase 3
JOB NO.: 60645745

TEST HOLE NO.:	**TH21-04**
SAMPLE NO.:	**T2C**
SAMPLE DEPTH:	1.52 - 2.13 m
DATE TESTED:	2-Feb-21
SHEAR STRENGTH TESTS	
TORVANE	
Reading	0.00
Vane Size (S, M, L)	M
Undrained Shear Strength (kPa)	**0.0**
Undrained Shear Strength (ksf)	0.00
POCKET PENETROMETER	
Reading - Qu (tsf)	0.00
Undrained Shear Strength (kPa)	**0.0**
Reading - Qu (tsf)	0.00
Undrained Shear Strength (kPa)	**0.0**
Reading - Qu (tsf)	0.00
Undrained Shear Strength (kPa)	**0.0**
UNCONFINED COMPRESSIVE STRENGTH TEST	
Unconfined compressive strength (kPa)	**48.5**
Unconfined compressive strength (ksf)	**1.0**
Undrained Shear Strength (kPa)	**24.3**
Undrained Shear Strength (ksf)	**0.507**
MOISTURE CONTENT	
Tare Number	T17
Wt. Sample wet + tare (g)	431.4
Wt. Sample dry + tare (g)	397.7
Wt. Tare (g)	8.8
Moisture Content %	**8.7**
BULK DENSITY	
Sample Wt. (g)	1500
Diameter 1 (cm)	7.20
Diameter 2 (cm)	7.20
Diameter 3 (cm)	7.30
Avg. Diameter (cm)	**7.23**
Length 1 (cm)	15.20
Length 2 (cm)	15.20
Length 3 (cm)	15.30
Avg. Length (cm)	**15.23**
Volume (cm^3)	626.0
Moisture content (%)	8.7
Bulk Density (g/cm^3)	2.396
Bulk Density (kN/m^3)	**23.5**
Bulk Density (pcf)	**149.6**
Dry Density (kN/m^3)	**21.63**

AECOM - SOILS LABORATORY
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS (ASTM D2166)

AECOM

CLIENT:	City of Winnipeg
PROJECT:	HRRC Phase 3
JOB NO.:	60645745

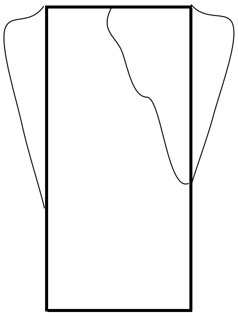
TEST HOLE NO.:	TH21-04
SAMPLE NO.:	T2
SAMPLE DEPTH:	1.52 - 2.13 m
SAMPLE DATE:	
TEST DATE:	2-Feb-21

SOIL DESCRIPTION:

SILT (Till) - Some clay, some sand, trace to some gravel, light brown,
moist, soft to firm, intermediate plasticity

MOISTURE CONTENT: 8.7

SAMPLE DIAM.(Do):	72.33	**(mm)**	**INITIAL AREA, Ao:**	4109.3	**(mm^2)**
SAMPLE LENGTH, (Lo):	152.33	**(mm)**	**PISTON RATE:**	0.0602	**(inches / minute)**
L / D RATIO:	2.11	**(2 < L/D < 2.5)**	**AXIAL STRAIN RATE, R:**	1.00	**(0.5<R<2 % / minute)**



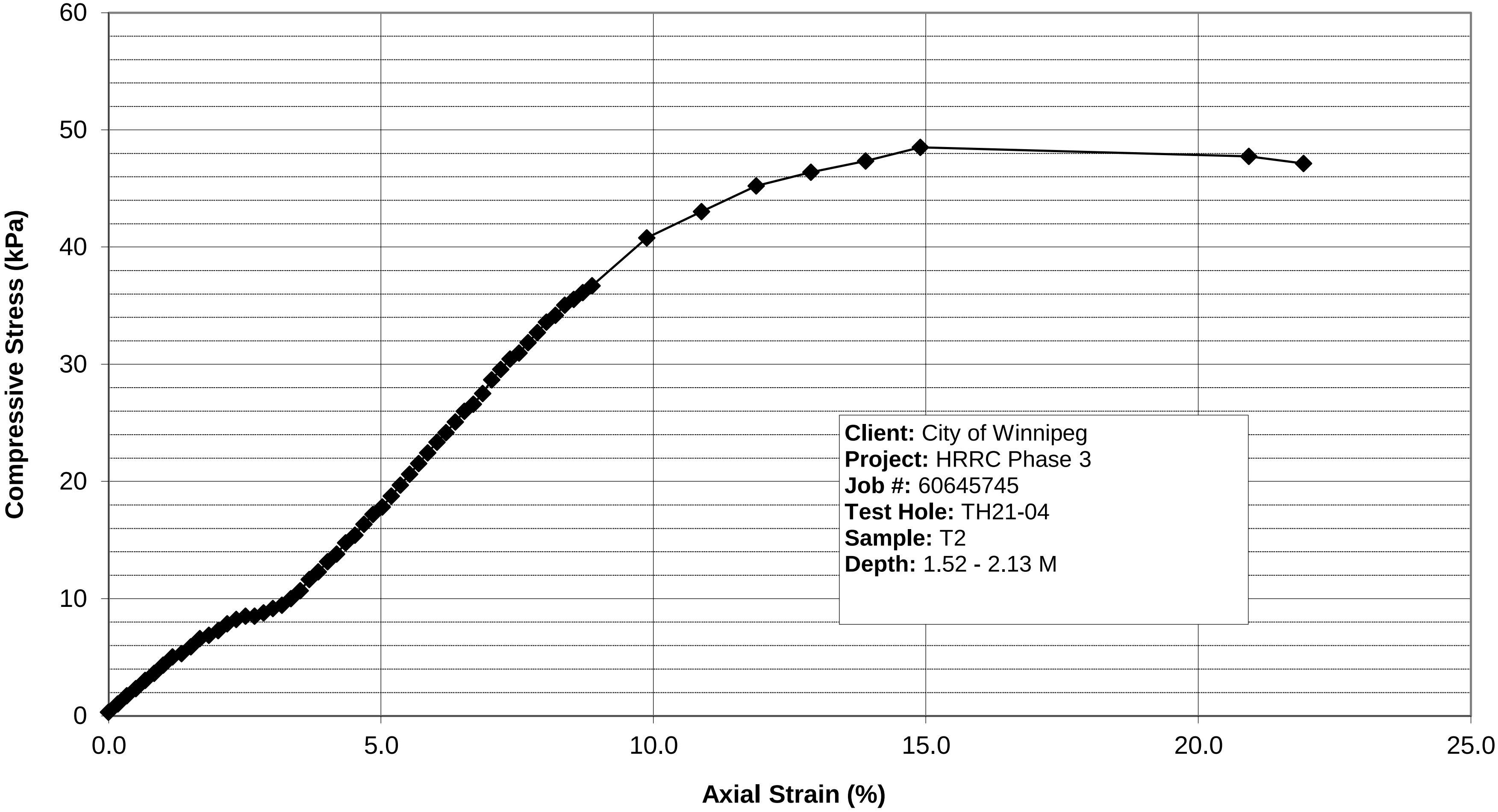
FAILURE SKETCH

TEST DATA - DIAL READINGS							
AXIAL COMPRESSION	PROVING RING	TOTAL AXIAL STRAIN, E_1	AVERAGE CROSS-SECTIONAL AREA, A	APPLIED AXIAL LOAD, P	COMPRESSIVE STRESS, σ_C		
(inches)	(inches)	(%)	(inches2)	(lbs)	(psi)	(ksf)	(kPa)
0.01	0.0000	0.00	6.37	0.28	0.04	0.006	0.3
0.02	0.0001	0.17	6.38	0.94	0.15	0.021	1.0
0.03	0.0002	0.33	6.39	1.59	0.25	0.036	1.7
0.04	0.0002	0.50	6.40	2.16	0.34	0.048	2.3
0.05	0.0003	0.67	6.41	2.81	0.44	0.063	3.0
0.06	0.0004	0.84	6.42	3.37	0.53	0.076	3.6
0.07	0.0004	1.00	6.43	4.03	0.63	0.090	4.3
0.08	0.0005	1.17	6.44	4.69	0.73	0.105	5.0
0.09	0.0005	1.34	6.46	4.97	0.77	0.111	5.3
0.10	0.0006	1.51	6.47	5.53	0.85	0.123	5.9
0.11	0.0007	1.67	6.48	6.18	0.95	0.137	6.6
0.12	0.0007	1.84	6.49	6.47	1.00	0.143	6.9
0.13	0.0007	2.01	6.50	6.84	1.05	0.152	7.3
0.14	0.0008	2.18	6.51	7.40	1.14	0.164	7.8
0.15	0.0008	2.34	6.52	7.78	1.19	0.172	8.2
0.16	0.0009	2.51	6.53	8.06	1.23	0.178	8.5
0.17	0.0009	2.68	6.54	8.06	1.23	0.177	8.5
0.18	0.0009	2.85	6.56	8.34	1.27	0.183	8.8
0.19	0.0009	3.01	6.57	8.71	1.33	0.191	9.1
0.20	0.0010	3.18	6.58	9.00	1.37	0.197	9.4
0.21	0.0010	3.35	6.59	9.56	1.45	0.209	10.0
0.22	0.0011	3.52	6.60	10.21	1.55	0.223	10.7
0.23	0.0012	3.68	6.61	11.15	1.69	0.243	11.6
0.24	0.0013	3.85	6.62	11.81	1.78	0.257	12.3
0.25	0.0014	4.02	6.64	12.65	1.91	0.274	13.1
0.26	0.0014	4.18	6.65	13.31	2.00	0.288	13.8
0.27	0.0015	4.35	6.66	14.24	2.14	0.308	14.7
0.28	0.0016	4.52	6.67	14.90	2.23	0.322	15.4
0.29	0.0017	4.69	6.68	15.84	2.37	0.341	16.3
0.30	0.0018	4.85	6.69	16.68	2.49	0.359	17.2
0.31	0.0019	5.02	6.71	17.33	2.58	0.372	17.8
0.32	0.0020	5.19	6.72	18.27	2.72	0.392	18.8
0.33	0.0021	5.36	6.73	19.21	2.85	0.411	19.7
0.34	0.0022	5.52	6.74	20.15	2.99	0.430	20.6
0.35	0.0023	5.69	6.75	21.08	3.12	0.450	21.5
0.36	0.0024	5.86	6.77	22.02	3.25	0.469	22.4
0.37	0.0025	6.03	6.78	22.96	3.39	0.488	23.4
0.38	0.0025	6.19	6.79	23.80	3.51	0.505	24.2
0.39	0.0026	6.36	6.80	24.74	3.64	0.524	25.1
0.40	0.0027	6.53	6.81	25.67	3.77	0.543	26.0
0.41	0.0028	6.70	6.83	26.33	3.86	0.555	26.6
0.42	0.0029	6.86	6.84	27.27	3.99	0.574	27.5
0.43	0.0030	7.03	6.85	28.48	4.16	0.599	28.7
0.44	0.0031	7.20	6.86	29.42	4.29	0.617	29.6
0.45	0.0032	7.37	6.88	30.36	4.42	0.636	30.4
0.46	0.0033	7.53	6.89	30.92	4.49	0.646	31.0
0.47	0.0034	7.70	6.90	31.86	4.62	0.665	31.8
0.48	0.0035	7.87	6.91	32.80	4.74	0.683	32.7
0.49	0.0036	8.03	6.93	33.73	4.87	0.701	33.6
0.50	0.0037	8.20	6.94	34.39	4.96	0.714	34.2
0.51	0.0038	8.37	6.95	35.32	5.08	0.732	35.0
0.52	0.0038	8.54	6.96	35.89	5.15	0.742	35.5
0.53	0.0039	8.70	6.98	36.54	5.24	0.754	36.1
0.54	0.0040	8.87	6.99	37.20	5.32	0.766	36.7
0.60	0.0045	9.88	7.07	41.79	5.91	0.851	40.8
0.66	0.0048	10.88	7.15	44.60	6.24	0.899	43.0
0.72	0.0051	11.89	7.23	47.41	6.56	0.945	45.2
0.78	0.0053	12.89	7.31	49.19	6.73	0.969	46.4
0.84	0.0054	13.89	7.40	50.79	6.87	0.989	47.3
0.90	0.0056	14.90	7.48	52.66	7.04	1.013	48.5
1.26	0.0060	20.92	8.05	55.75	6.92	0.997	47.7
1.33	0.0060	21.93	8.16	55.75	6.83	0.984	47.1

UNCONFINED COMPRESSIVE STRENGTH, q_u:	48.51	kPa
(based on maximum q_u value)	1.013	ksf
UNDRAINED SHEAR STRENGTH, S_u:	24.26	kPa
(based on maximum q_u value)	0.507	ksf

NOTES:

AECOM
UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
(ASTM D2166)



Client: City of Winnipeg
Project: HRRC Phase 3
Job #: 60645745
Test Hole: TH21-04
Sample: T2
Depth: 1.52 - 2.13 M

AECOM Canada Ltd.
ATTN: RYAN HARRAS
99 Commerce Drive
Winnipeg MB R3P 0Y7

Date Received: 05-FEB-21
Report Date: 16-FEB-21 07:10 (MT)
Version: FINAL

Client Phone: 204-477-5381

Certificate of Analysis

Lab Work Order #: L2555270

Project P.O. #: 60645745
Job Reference: 60645745
C of C Numbers:
Legal Site Desc:

Hua Wo
Chemistry Laboratory Manager

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters		Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2555270-1	TH21-01; G1 @ 2.5’							
Sampled By:	CLIENT							
Matrix:	SOIL							
Miscellaneous Parameters								
% Moisture		18.0		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride		373		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity		1210		1.0	ohm*cm		12-FEB-21	
Sulphate		35		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity		0.824		0.0040	mS/cm		12-FEB-21	R5374140
pH		7.49		0.10	pH units		10-FEB-21	R5369804
L2555270-2	TH21-01; G5 @ 12.5’							
Sampled By:	CLIENT							
Matrix:	SOIL							
Miscellaneous Parameters								
% Moisture		20.5		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride		306		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity		1330		1.0	ohm*cm		11-FEB-21	
Sulphate		118		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity		0.750		0.0040	mS/cm		11-FEB-21	R5372222
pH		7.76		0.10	pH units		10-FEB-21	R5369804
L2555270-3	TH21-01; S8 @ 20’							
Sampled By:	CLIENT							
Matrix:	SOIL							
Miscellaneous Parameters								
% Moisture		9.64		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride		132		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity		2420		1.0	ohm*cm		11-FEB-21	
Sulphate		76		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity		0.414		0.0040	mS/cm		11-FEB-21	R5372222
pH		8.10		0.10	pH units		10-FEB-21	R5369804
L2555270-4	TH21-02; G1 @ 2.5’							
Sampled By:	CLIENT							
Matrix:	SOIL							
Miscellaneous Parameters								
% Moisture		19.3		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride		64		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity		1940		1.0	ohm*cm		11-FEB-21	
Sulphate		58		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity		0.515		0.0040	mS/cm		11-FEB-21	R5372222
pH		7.65		0.10	pH units		10-FEB-21	R5369804
L2555270-5	TH21-02; G3 @ 7.5’							
Sampled By:	CLIENT							
Matrix:	SOIL							
Miscellaneous Parameters								
% Moisture		26.5		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride		116		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity		1710		1.0	ohm*cm		11-FEB-21	
Sulphate		128		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity		0.584		0.0040	mS/cm		11-FEB-21	R5372222
pH		7.67		0.10	pH units		10-FEB-21	R5369804

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2555270-6 TH21-02; S6 @ 14’							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	10.7		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	120		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	1700		1.0	ohm*cm		11-FEB-21	
Sulphate	177		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.587		0.0040	mS/cm		11-FEB-21	R5372222
pH	8.03		0.10	pH units		10-FEB-21	R5369804
L2555270-7 TH21-03; G1 @ 2.5’							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	17.9		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	32		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	2400		1.0	ohm*cm		11-FEB-21	
Sulphate	21		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.416		0.0040	mS/cm		11-FEB-21	R5372222
pH	7.44		0.10	pH units		10-FEB-21	R5369804
L2555270-8 TH21-03; S4 @ 10’							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	8.36		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	35		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	2860		1.0	ohm*cm		12-FEB-21	
Sulphate	192		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.350		0.0040	mS/cm		12-FEB-21	R5374140
pH	8.14		0.10	pH units		10-FEB-21	R5369804
L2555270-9 TH21-03; G7 @ 17.5’							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	7.32		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	21		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	3190		1.0	ohm*cm		12-FEB-21	
Sulphate	112		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.313		0.0040	mS/cm		12-FEB-21	R5374140
pH	8.10		0.10	pH units		10-FEB-21	R5369804
L2555270-10 TH21-04; G1 @ 2.5’							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	26.7		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	<20		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	2040		1.0	ohm*cm		12-FEB-21	
Sulphate	126		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.489		0.0040	mS/cm		12-FEB-21	R5374140
pH	7.83		0.10	pH units		10-FEB-21	R5369804

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2555270-11 TH21-04; S4 @ 10'							
Sampled By: CLIENT							
Matrix: SOIL							
Miscellaneous Parameters							
% Moisture	10.2		0.25	%	10-FEB-21	11-FEB-21	R5369305
Chloride	27		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Resistivity	3790		1.0	ohm*cm		12-FEB-21	
Sulphate	62		20	mg/kg	10-FEB-21	10-FEB-21	R5371260
Conductivity	0.264		0.0040	mS/cm		12-FEB-21	R5374140
pH	8.03		0.10	pH units		10-FEB-21	R5369798

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-WT	Soil	Chloride in Soil	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

ALS Test Code	Matrix	Test Description	Method Reference**
EC-WT	Soil	Conductivity (EC)	MOEE E3138

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

ALS Test Code	Matrix	Test Description	Method Reference**
MOISTURE-WT	Soil	% Moisture	CCME PHC in Soil - Tier 1 (mod)
PH-WT	Soil	pH	MOEE E3137A

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

ALS Test Code	Matrix	Test Description	Method Reference**
RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

"Soil Resistivity (calculated)" is determined as the inverse of the conductivity of a 2:1 water:soil leachate (dry weight). This method is intended as a rapid approximation for Soil Resistivity. Where high accuracy results are required, direct measurement of Soil Resistivity by the Wenner Four-Electrode Method (ASTM G57) is recommended.

ALS Test Code	Matrix	Test Description	Method Reference**
SO4-WT	Soil	Sulphate	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid-adjusted weight
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L2555270 Report Date: 16-FEB-21

Client: AECOM Canada Ltd.
99 Commerce Drive
Winnipeg MB R3P 0Y7
Contact: RYAN HARRAS

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-WT	**Soil**							
Batch R5371260								
WG3486087-4 CRM		**AN-CRM-WT**						
Chloride			99.8		%		70-130	10-FEB-21
WG3486087-2 LCS								
Chloride			99.1		%		80-120	10-FEB-21
WG3486087-1 MB								
Chloride			<20		mg/kg		20	10-FEB-21
EC-WT	**Soil**							
Batch R5372222								
WG3486698-2 IRM		**WT SAR4**						
Conductivity			106.0		%		70-130	11-FEB-21
WG3487076-1 LCS								
Conductivity			102.3		%		90-110	11-FEB-21
WG3486698-1 MB								
Conductivity			<0.0040		mS/cm		0.004	11-FEB-21
Batch R5374140								
WG3487289-2 IRM		**WT SAR4**						
Conductivity			104.8		%		70-130	12-FEB-21
WG3487666-1 LCS								
Conductivity			99.0		%		90-110	12-FEB-21
WG3487289-1 MB								
Conductivity			<0.0040		mS/cm		0.004	12-FEB-21
MOISTURE-WT	**Soil**							
Batch R5369305								
WG3486090-2 LCS								
% Moisture			99.5		%		90-110	11-FEB-21
WG3486090-1 MB								
% Moisture			<0.25		%		0.25	11-FEB-21
PH-WT	**Soil**							
Batch R5369798								
WG3486215-1 LCS								
pH			6.99		pH units		6.9-7.1	10-FEB-21
Batch R5369804								
WG3486214-1 LCS								
pH			6.99		pH units		6.9-7.1	10-FEB-21
SO4-WT	**Soil**							

Quality Control Report

Test		Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
SO4-WT		**Soil**							
Batch	**R5371260**								
WG3486087-4	**CRM**		**AN-CRM-WT**						
Sulphate				103.4		%		60-140	10-FEB-21
WG3486087-2	**LCS**								
Sulphate				99.4		%		80-120	10-FEB-21
WG3486087-1	**MB**								
Sulphate				<20		mg/kg		20	10-FEB-21

Quality Control Report

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

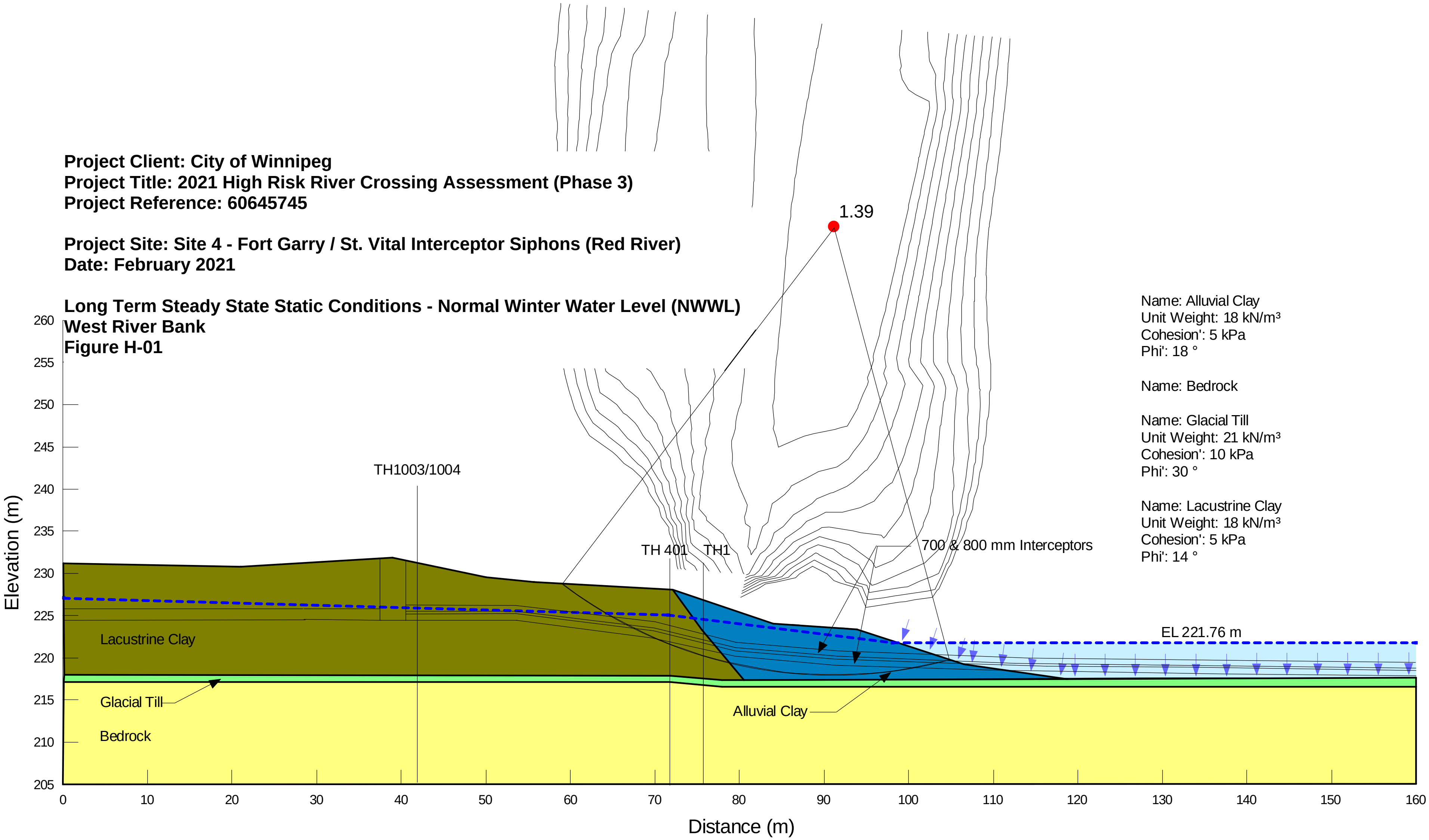
Appendix H

Slope Stability Analysis Output

Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 4 - Fort Garry / St. Vital Interceptor Siphons (Red River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Winter Water Level (NWWL)
West River Bank
Figure H-01



Name: Alluvial Clay
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 18 °

Name: Bedrock

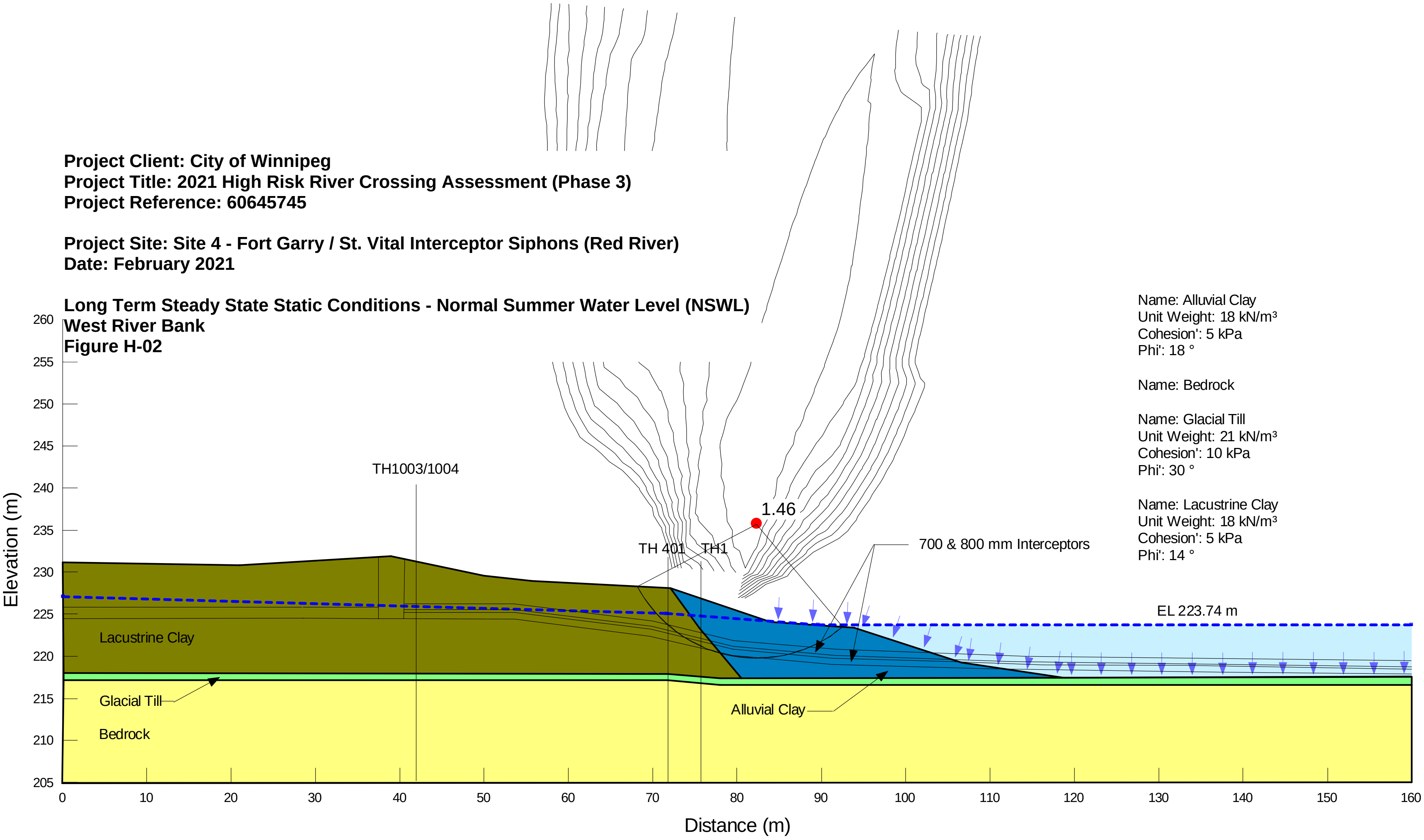
Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 10 kPa
Phi': 30 °

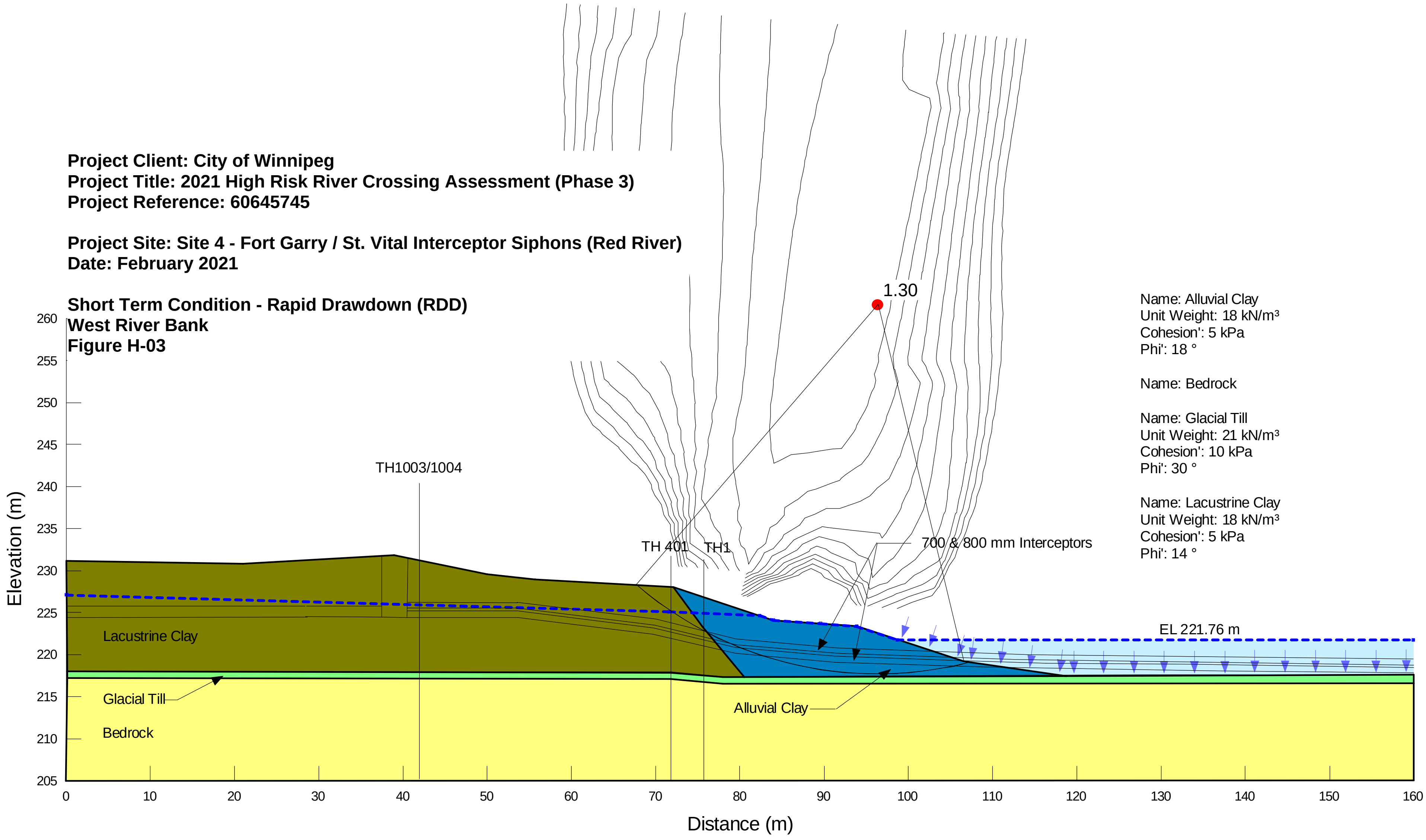
Name: Lacustrine Clay
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 4 - Fort Garry / St. Vital Interceptor Siphons (Red River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Summer Water Level (NSWL)
West River Bank
Figure H-02





Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 4 - Fort Garry / St. Vital Interceptor Siphons (Red River)
Date: February 2021

Short Term Condition - Rapid Drawdown (RDD)
West River Bank
Figure H-03

Name: Alluvial Clay
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 18 °

Name: Bedrock

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 10 kPa
Phi': 30 °

Name: Lacustrine Clay
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 14 °

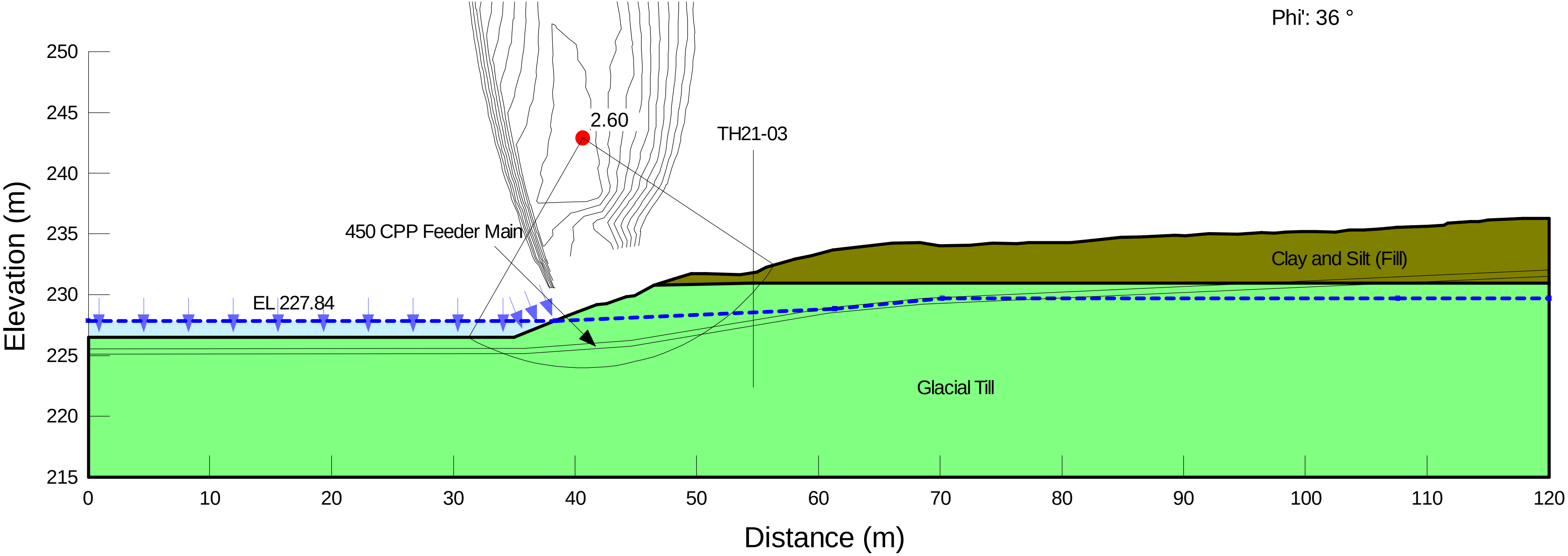
Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Winter Water Level (NWWL)
North River Bank
Figure H-04

Name: Clay and Silt (Fill)
Unit Weight: 18.5 kN/m³
Cohesion': 2 kPa
Phi': 18 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °



Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

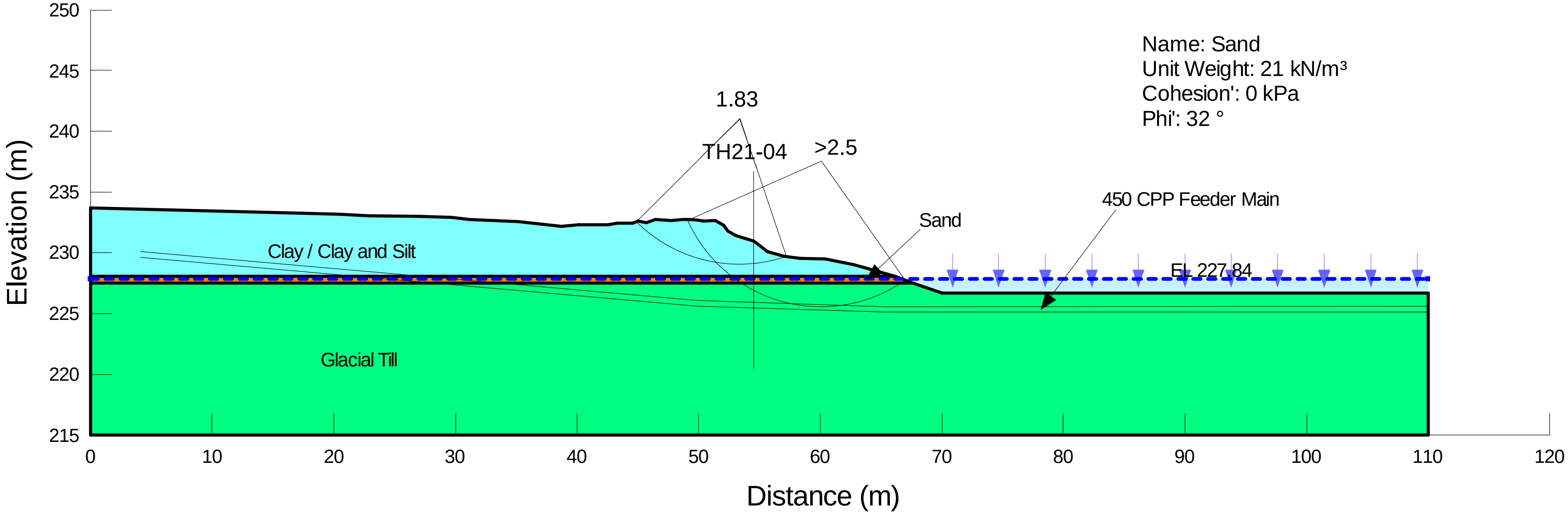
Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Winter Water Level (NWWL)
South River Bank
Figure H-05

Name: Clay / Clay and Silt
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °

Name: Sand
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 32 °



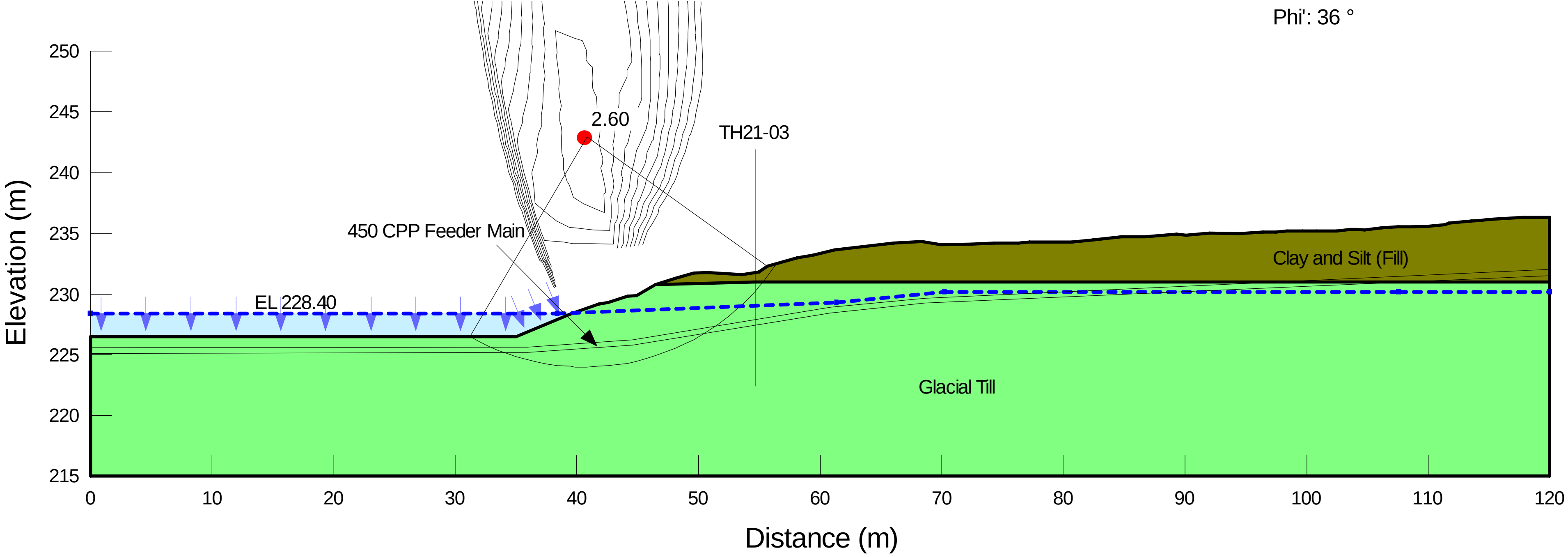
Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Summer Water Level (NSWL)
North River Bank
Figure H-06

Name: Clay and Silt (Fill)
Unit Weight: 18.5 kN/m³
Cohesion': 2 kPa
Phi': 18 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °



Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

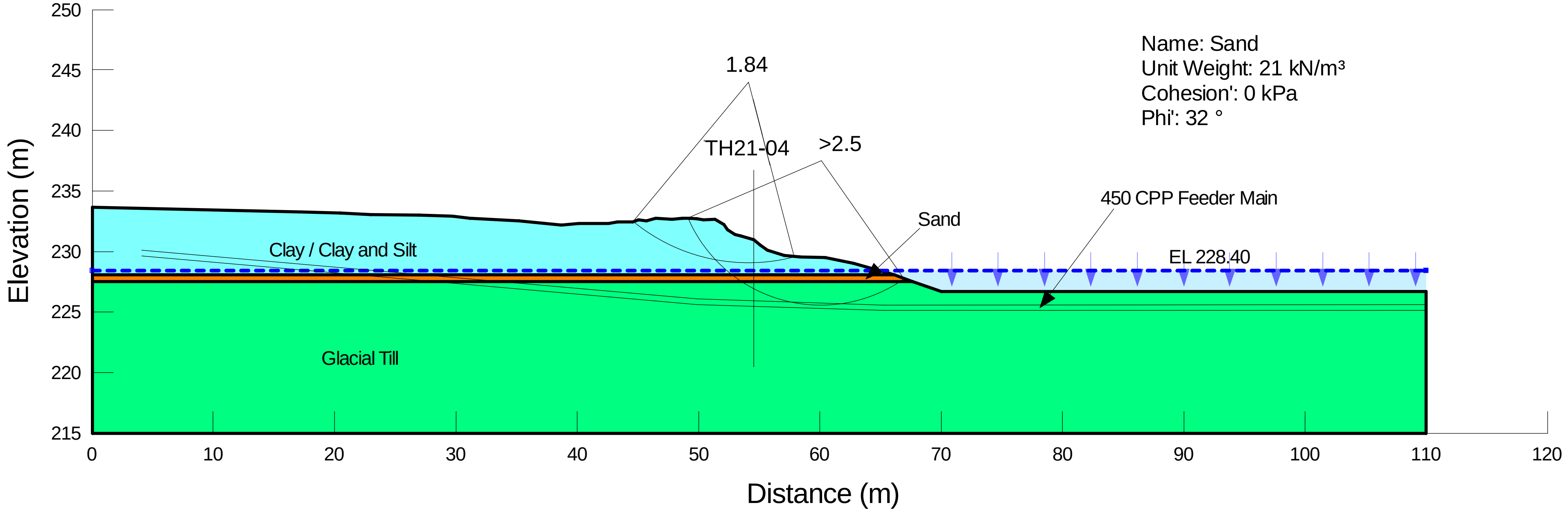
Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Long Term Steady State Static Conditions - Normal Summer Water Level (NSWL)
South River Bank
Figure H-07

Name: Clay / Clay and Silt
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °

Name: Sand
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 32 °



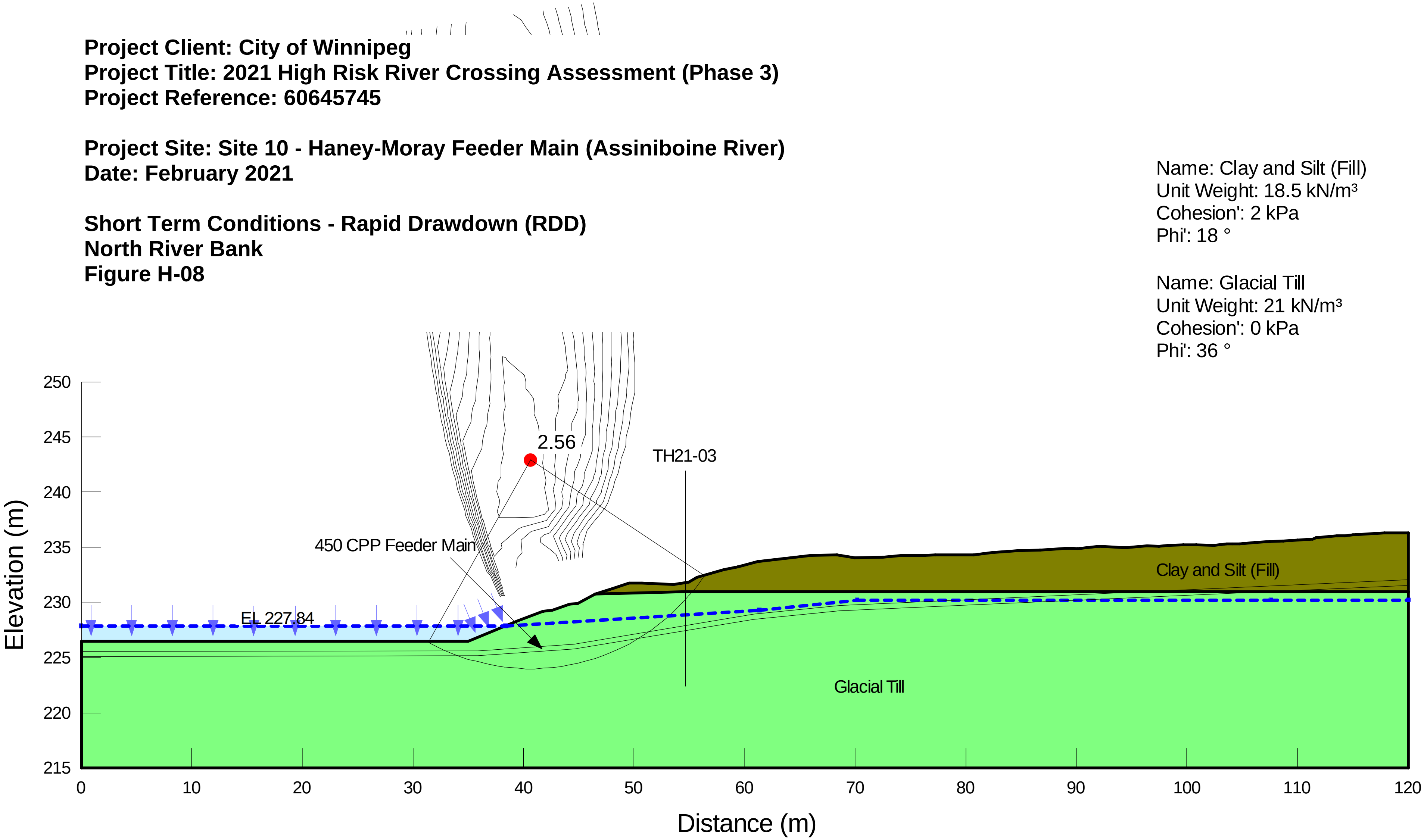
Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Short Term Conditions - Rapid Drawdown (RDD)
North River Bank
Figure H-08

Name: Clay and Silt (Fill)
Unit Weight: 18.5 kN/m³
Cohesion': 2 kPa
Phi': 18 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °



Project Client: City of Winnipeg
Project Title: 2021 High Risk River Crossing Assessment (Phase 3)
Project Reference: 60645745

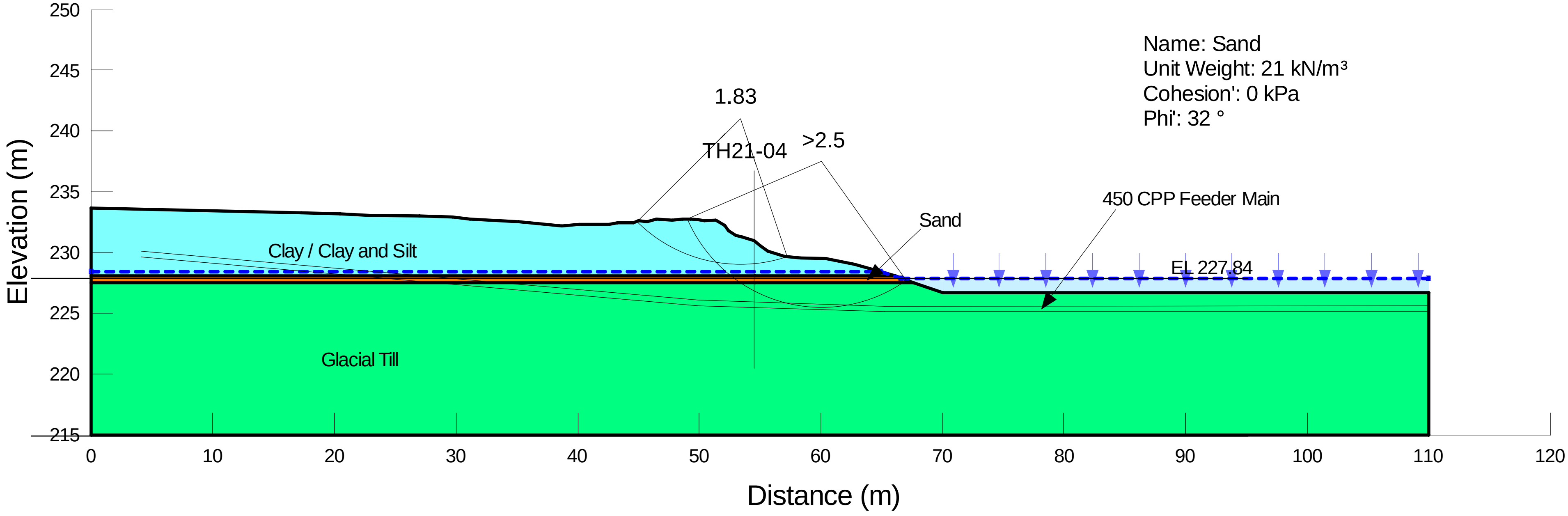
Project Site: Site 10 - Haney-Moray Feeder Main (Assiniboine River)
Date: February 2021

Short Term Conditions - Rapid Drawdown (RDD)
South River Bank
Figure H-09

Name: Clay / Clay and Silt
Unit Weight: 18 kN/m³
Cohesion': 5 kPa
Phi': 14 °

Name: Glacial Till
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 36 °

Name: Sand
Unit Weight: 21 kN/m³
Cohesion': 0 kPa
Phi': 32 °



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Appendix II

Environmental Results

Table 1: Summary of Groundwater Analytical Results - Volatile Organic Compound and Petroleum Hydrocarbon Parameters

					Sample ID	TH24-01	TH24-02	DUP-01
					Sample Date	6-Feb-25	6-Feb-25	6-Feb-25
					Screen interval (mbgs)	18.3 - 25.2	15.5 - 24.7	15.5 - 24.7
					Lab sample ID	WP2501636-001	WP2501636-002	WP2501636-003
					Lab work order	WP2501636	WP2501636	WP2501636
					Sample type	N	N	FD
Parameter	**Units**	**Minimum RDL**	**Winnipeg By-Law Schedule B[1]**	**Winnipeg By-Law Schedule D[2]**	**Surface Water FAL WQG[3]**	**Analytical Results**		
Benzene	mg/L	0.00050	0.5	0.002	0.37	<0.00050	<0.00050	<0.00050
Ethylbenzene	mg/L	0.00050	0.024	0.002	0.002	<0.00050	<0.00050	<0.00050
Toluene	mg/L	0.00050	NG	NG	NG	<0.00050	<0.00050	<0.00050
Xylene, m+p-	mg/L	0.00040	NG	NG	NG	<0.00040	<0.00040	<0.00040
Xylene, o-	mg/L	0.00030	NG	NG	NG	<0.00030	<0.00030	<0.00030
Xylenes, total	mg/L	0.00050	1.4	0.044	NG	<0.00050	<0.00050	<0.00050
Styrene	mg/L	0.00050	NG	NG	NG	<0.00050	<0.00050	<0.00050
PHC F1 (C6-C10) minus BTEX	mg/L	0.10	NG	NG	NG	<0.10	<0.10	<0.10
PHC F2 (>C10-C16)	mg/L	0.10	NG	NG	NG	<0.10	<0.10	<0.10
PHC F2 (>C10-C16)	mg/L	0.10	NG	NG	NG	<0.10	<0.10	<0.10

Guidelines:

[1] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges into Wastewater System (2022)

[2] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges to Land Drainage Systemn (2022)

[3] Manitoba Tier III Water Quality Guidelines for Surface Water: Freshwater Aquatic Life, Manitoba Water Quality Standards, Objectives, and Guidelines (2011)

Notes:

BOLD = value exceeds Wastewater By-Law Schedule B Guideline
<u>BOLD</u> = value exceeds Wastewater By-Law Schedule D Guideline
BOLD = value exceeds MB Tier III Water Quality Guideline
\- = no data
mbgs = metres below ground surface
NG = No Guideline
N/A = Not Applicable
RDL = Reportable Detection Limit
GWQG = Groundwater Quality Guidelines
See laboratory certificates for additonal analysis qualifiers

Table 2: Summary of Groundwater Analytical Results - Polycyclic Aromatic Hydrocarbon Parameters

					Sample ID	**TH24-01**	**TH24-05**	**DUP-01**
					Sample Date	6-Feb-25	6-Feb-25	6-Feb-25
					Screen interval (mbgs)	**18.3 - 25.2**	**15.5 - 24.7**	**15.5 - 24.7**
					Lab sample ID	**WP2501636-001**	**WP2501636-002**	**WP2501636-003**
					Lab work order	**WP2501636**	**WP2501636**	**WP2501636**
					Sample type	**N**	**FD**	**FD**
Parameter	**Units**	**Minimum RDL**	**Winnipeg By-Law Schedule B[1]**	**Winnipeg By-Law Schedule D[2]**	**Surface Water FAL WQG[3]**	**Analytical Results (µg/L)**		
Acenaphthene	µg/L	0.010	NG	NG	5800	<0.010	<0.019	<0.010
Acenaphthylene	µg/L	0.010	NG	NG	NG	<0.010	<0.010	<0.010
Acridine	µg/L	0.010	NG	NG	4400	<0.016	<0.091	<0.039
Anthracene	µg/L	0.010	NG	NG	12	<0.010	<0.020	<0.010
Benz(a)anthracene	µg/L	0.010	NG	NG	18	<0.010	<0.010	<0.010
Benzo(a)pyrene	µg/L	0.0050	NG	NG	15	<0.0050	<0.0050	<0.0050
Benzo(b+j)fluoranthene	µg/L	0.010	NG	NG	NG	<0.010	<0.015	<0.010
Benzo(b+j+k)fluoranthene	µg/L	0.015	NG	NG	NG	<0.015	<0.021	<0.015
Benzo(g,h,i)perylene	µg/L	0.010	NG	NG	NG	<0.010	<0.010	<0.010
Benzo(k)fluoranthene	µg/L	0.010	NG	NG	NG	<0.010	<0.015	<0.010
Chrysene	µg/L	0.010	NG	NG	NG	<0.010	<0.014	<0.010
Dibenz(a,h)anthracene	µg/L	0.0050	NG	NG	NG	<0.0050	<0.0050	<0.0050
Fluoranthene	µg/L	0.010	NG	NG	40	0.015	<0.056	0.026
Fluorene	µg/L	0.010	NG	NG	3000	<0.010	0.035	0.016
Indeno(1,2,3-c,d)pyrene	µg/L	0.010	NG	NG	NG	<0.010	<0.010	<0.010
Methylnaphthalene, 1+2-	µg/L	0.015	NG	NG	NG	0.059	0.127	0.063
Methylnaphthalene, 1-	µg/L	0.010	NG	NG	NG	0.024	0.050	0.025
Methylnaphthalene, 2-	µg/L	0.010	NG	NG	NG	0.035	0.077	0.038
Naphthalene	µg/L	0.050	NG	NG	1100	<0.050	0.059	<0.050
Phenanthrene	µg/L	0.020	NG	NG	400	0.025	0.099	0.040
Pyrene	µg/L	0.010	NG	NG	25	0.030	0.095	0.050
Quinoline	µg/L	0.050	NG	NG	3400	<0.050	<0.050	<0.050
B(a)P total potency equivalents [B(a)P TPE]	µg/L	0.010	NG	NG	NG	<0.010	<0.010	<0.010
PAHs, high molecular weight (BC AWQ)	µg/L	0.030	NG	NG	NG	0.045	0.095	0.076
PAHs, low molecular weight (BC AWQ)	µg/L	0.060	NG	NG	NG	<0.060	0.193	<0.060
PAHs, total (CCME sewer 18)	µg/L	0.070	NG	NG	NG	0.129	0.415	0.195
PAHs, total (EPA 16)	µg/L	0.065	5	2	NG	0.070	0.288	0.132

Guidelines:

[1] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges into Wastewater System (2022)

[2] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for

[3] Manitoba Tier III Water Quality Guidelines for Surface Water: Freshwater Aquatic Life, Manitoba Water Quality Standards, Objectives, and Guidelines (2011)

Notes:

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mbgs = metres below ground surface
NG = No Guideline
N/A = Not Applicable
RDL = Reportable Detection Limit
GWQG = Groundwater Quality Guidelines

See laboratory certificates for additonal analysis qualifiers

Table 3: Summary of Groundwater Analytical Results - Nutrient Parameters

					Sample ID	**TH24-01**	**TH24-05**	**DUP-01**
					Date	**6-Feb-25**	**6-Feb-25**	**6-Feb-25**
					Screen interval (mbgs)	**18.3 - 25.2**	**15.5 - 24.7**	**15.5 - 24.7**
					Lab sample ID	**WP2501636-001**	**WP2501636-002**	**WP2501636-003**
					Lab work order	**WP2501636**	**WP2501636**	**WP2501636**
					Sample type	**N**	**N**	**FD**
Parameter	**Units**	**Minimum RDL**	**Winnipeg By-Law Schedule B^1**	**Winnipeg By-Law Schedule D^2**	**Surface Water FAL WQG^3**	**Analytical Results**		
Nutrients								
Ammonia, total (as N)	mg/L	0.0050	NG	NG	NG	0.891	0.882	0.882
Nitrate (as N)	mg/L	0.020	NG	NG	13	<1.00	<0.400	<0.400
Nitrite (as N)	mg/L	0.010	NG	NG	0.197	<0.500	<0.200	<0.200
Total Nitrogen	mg/L	0.5	60	NG	NG	1.641	1.182	1.182
Calcium (Dissolved)	mg/L	0.050	NG	NG	NG	257	239	245
Calcium (Total)	mg/L	0.050	NG	NG	NG	1070	4730	4180
Magnesium (Dissolved)	mg/L	0.0050	NG	NG	NG	137	151	144
Magnesium (Total)	mg/L	0.0050	NG	NG	NG	587	2400	2260
Phosphorus (Dissolved)	mg/L	0.050	NG	NG	Variable ^{c}	<0.050	<0.050	<0.050
Phosphorus (Total)	mg/L	0.050	10	0.4	NG	3.96	**14.2**	**12.6**
Potassium (Dissolved)	mg/L	0.050	NG	NG	NG	41.9	33.6	33.9
Potassium (Total)	mg/L	0.050	NG	NG	NG	73.1	114	103
Sodium (Dissolved)	mg/L	0.050	NG	NG	NG	1110	935	921
Sodium (Total)	mg/L	0.050	NG	NG	NG	1090	959	870

Guidelines:

[1] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges into Wastewater System (2022)

[2] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges to Land Drainage Systemn (2022)

[3] Manitoba Tier III Water Quality Guidelines for Surface Water: Freshwater Aquatic Life, Manitoba Water Quality Standards, Objectives, and Guidelines (2011)

Notes:

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BOLD = value exceeds MB Tier III Water Quality Guideline
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mbgs = metres below ground surface
NG = No Guideline
RDL = Reportable Detection Limit
GWQG = Groundwater Quality Guidelines
See laboratory certificates for additonal analysis qualifiers

Table 4: Summary of Groundwater Analytical Results - Dissolved Metal Parameters

					Sample ID	TH24-01	TH24-05	DUP-01
					Date	6-Feb-25	6-Feb-25	6-Feb-25
					Screen interval (mbgs)	18.3 - 25.2	15.5 - 24.7	15.5 - 24.7
					Lab sample ID	WP2501636-001	WP2501636-002	WP2501636-003
					Lab work order	WP2501636	WP2501636	WP2501636
					Sample type	N	N	FD
Parameter	**Units**	**Minimum RDL**	**Winnipeg By-Law Schedule B[1]**	**Winnipeg By-Law Schedule D[2]**	**Surface Water FAL WQG[3]**	**Analytical Results**		
Aluminum	mg/L	0.0010	NG	NG	0.005^{A} or 0.1^{B}	0.0015	0.0016	0.0014
Antimony	mg/L	0.00010	NG	NG	NG	<0.00010	0.00011	0.00012
Arsenic	mg/L	0.00010	NG	NG	5	0.00087	0.00076	0.00060
Barium	mg/L	0.00010	NG	NG	NG	0.0217	0.0321	0.0344
Beryllium	mg/L	0.000020	NG	NG	NG	<0.000020	<0.000020	<0.000020
Bismuth	mg/L	0.000050	NG	NG	NG	<0.000050	<0.000050	<0.000050
Boron	µg/L	0.010	NG	NG	1.5	0.809	0.792	0.824
Cadmium	mg/L	0.0000050	NG	NG	0.00004^{c}	<0.0000050	0.0000111	0.0000111
Cesium	mg/L	0.000010	NG	NG	NG	0.000050	0.000070	0.000070
Chromium	mg/L	0.00050	NG	NG	NG	<0.00050	<0.00050	<0.00050
Cobalt	mg/L	0.00010	NG	NG	NG	0.00137	0.00097	0.00090
Copper	mg/L	0.00020	NG	NG	0.002^{c}	<0.00020	<0.00020	<0.00020
Iron	mg/L	0.010	NG	NG	300	<0.010	<0.010	<0.010
Lead	mg/L	0.000050	NG	NG	0.001^{c}	<0.000050	<0.000050	<0.000050
Lithium	mg/L	0.0010	NG	NG	NG	0.262	0.266	0.258
Manganese	mg/L	0.00010	NG	NG	Variable ^{c}	0.103	0.0941	0.0938
Molybdenum	mg/L	0.000050	NG	NG	73	0.00743	0.00386	0.00445
Nickel	mg/L	0.00050	NG	NG	0.025^{c}	0.00401	0.00377	0.00354
Rubidium	mg/L	0.00020	NG	NG	NG	0.0214	0.0131	0.0138
Selenium	mg/L	0.000050	NG	NG	1	<0.000050	<0.000050	<0.000050
Silicon	mg/L	0.050	NG	NG	NG	4.24	6.31	6.30
Silver	mg/L	0.000010	NG	NG	0.25	<0.000010	<0.000010	<0.000010
Strontium	mg/L	0.00020	NG	NG	NG	3.53	3.02	3.06
Sulfur	mg/L	0.50	NG	NG	NG	347	297	311
Tellurium	mg/L	0.00020	NG	NG	NG	<0.00020	<0.00020	<0.00020
Thallium	mg/L	0.000010	NG	NG	0.8	0.000010	0.000022	0.000036
Thorium	mg/L	0.00010	NG	NG	NG	<0.00010	<0.00010	<0.00010
Tin	mg/L	0.00010	NG	NG	NG	0.00194	0.00070	0.00077
Titanium	mg/L	0.00030	NG	NG	NG	<0.00030	<0.00030	<0.00030
Tungsten	mg/L	0.00010	NG	NG	NG	0.00042	0.00072	0.00098
Uranium	mg/L	0.000010	NG	NG	15	0.00247	0.00243	0.00259
Vanadium	mg/L	0.00050	NG	NG	NG	<0.00050	<0.00050	<0.00050
Zinc	mg/L	0.0010	NG	NG	Variable ^{c}	0.0015	0.0021	0.0015
Zirconium	mg/L	0.00030	NG	NG	NG	<0.00030	<0.00030	<0.00030

Guidelines:

[1] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges into Wastewater System (2022)

[2] The City of Winnipeg Sewer By-Law No. 106/2018 Schedule B Concentration Limits for Discharges to Land Drainage Systemn (2022)

[3] Manitoba Tier III Water Quality Guidelines for Surface Water: Freshwater Aquatic Life, Manitoba Water Quality Standards, Objectives, and Guidelines (2011)

Notes:

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BOLD	= value exceeds MB Tier III Water Quality Guideline
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mbgs	= metres below ground surface
NG	= No Guideline
N/A	= Not Applicable
RDL	= Reportable Detection Limit
A	= If pH is < 6.5
B	= If pH is ≥ 6.5
C	= Calculated guideline based on water hardness and/or other water quality parameters

See laboratory certificates for additonal analysis qualifiers

Table 5: Summary of Groundwater Analytical Results - Total Metal Parameters

					Sample ID	TH24-01	TH24-05	DUP-01
					Date	6-Feb-25	6-Feb-25	6-Feb-25
					Screen interval (mbgs)	18.3 - 25.2	15.5 - 24.7	15.5 - 24.7
					Lab sample ID	WP2501636-001	WP2501636-002	WP2501636-003
					Lab work order	WP2501636	WP2501636	WP2501636
					Sample type	N	N	FD
Parameter	**Units**	**Minimum RDL**	**Winnipeg By-Law Schedule B[1]**	**Winnipeg By-Law Schedule D[2]**	**Surface Water FAL WQG[3]**	**Analytical Results**		
Aluminum	mg/L	0.0030	50	NG	NG	***108***	***219***	***203***
Antimony	mg/L	0.00010	5	NG	NG	0.00198	0.00293	0.00299
Arsenic	mg/L	0.00010	1	0.02	NG	0.0778	**0.171**	**0.153**
Barium	mg/L	0.00010	NG	NG	NG	1.31	3.64	3.23
Beryllium	mg/L	0.000020	NG	NG	NG	0.00623	0.0173	0.0156
Bismuth	mg/L	0.000050	NG	NG	NG	0.00179	0.00544	0.00457
Boron	µg/L	0.010	NG	NG	NG	1.18	1.43	1.42
Cadmium	mg/L	0.0000050	0.7	0.008	NG	0.00228	0.00942	0.00714
Cesium	mg/L	0.000010	NG	NG	NG	0.0201	0.0488	0.0423
Chromium	mg/L	0.00050	4	0.08	NG	**0.234**	**0.889**	**0.706**
Cobalt	mg/L	0.00010	5	NG	NG	0.0766	0.240	0.181
Copper	mg/L	0.00050	2	0.04	NG	**0.219**	**0.747**	**0.584**
Iron	mg/L	0.010	NG	NG	NG	187	583	473
Lead	mg/L	0.000050	1	0.08	NG	**0.0944**	**0.276**	**0.235**
Lithium	mg/L	0.0010	NG	NG	NG	0.469	0.775	0.660
Manganese	mg/L	0.00010	5	0.2	NG	**3.00**	***17.8***	***12.2***
Molybdenum	mg/L	0.000050	5	NG	NG	0.0142	0.0291	0.0283
Nickel	mg/L	0.00050	2.0	0.08	NG	**0.228**	**0.714**	**0.544**
Rubidium	mg/L	0.00020	NG	NG	NG	0.273	0.666	0.552
Selenium	mg/L	0.000050	1	0.02	NG	0.00215	0.00749	0.00631
Silicon	mg/L	0.10	NG	NG	NG	236	468	451
Silver	mg/L	0.000010	5	0.04	NG	0.000698	0.00288	0.00234
Strontium	mg/L	0.00020	NG	NG	NG	4.38	6.92	5.48
Sulfur	mg/L	0.50	NG	NG	NG	342	308	293
Tellurium	mg/L	0.00020	NG	NG	NG	<0.00200	<0.00200	<0.00200
Thallium	mg/L	0.000010	NG	NG	NG	0.00203	0.00617	0.00522
Thorium	mg/L	0.00010	NG	NG	NG	0.0435	0.126	0.106
Tin	mg/L	0.00010	5	NG	NG	0.0583	0.0268	0.0316
Titanium	mg/L	0.00030	5	NG	NG	2.05	2.96	2.88
Tungsten	mg/L	0.00010	NG	NG	NG	0.00288	0.00803	0.00895
Uranium	mg/L	0.000010	NG	NG	NG	0.0137	0.0511	0.0400
Vanadium	mg/L	0.00050	NG	NG	NG	0.310	0.684	0.627
Zinc	mg/L	0.0030	2	0.04	NG	**0.756**	***2.68***	**1.98**
Zirconium	mg/L	0.00020	NG	NG	NG	0.0150	0.0111	0.0109

Guidelines:

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TEXT
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Table 6: Quality Assurance and Quality Control Results

			Sample ID	TH24-05	DUP-01	Greater Than 5x RDL	RPD (%)	Pass/Fail
			Date	6-Feb-25	6-Feb-25			
			Screen interval (mbgs)	15.5 - 24.7	15.5 - 24.7			
			Lab sample ID	WP2501636-002	WP2501636-003			
			Lab work order	WP2501636	WP2501636			
			Sample type	N	FD			
Parameter	Units	Minimum RDL	RPD Threshold (%)					
Aluminum (Total)	mg/L	0.0030	20	219	203	Yes	4.99	Pass
Antimony (Total)	mg/L	0.00010	20	0.00293	0.00299	Yes	1.36	Pass
Arsenic (Total)	mg/L	0.00010	20	0.171	0.153	Yes	7.27	Pass
Barium (Total)	mg/L	0.00010	20	3.64	3.23	Yes	7.80	Pass
Beryllium (Total)	mg/L	0.000020	20	0.0173	0.0156	Yes	6.77	Pass
Bismuth (Total)	mg/L	0.000050	20	0.00544	0.00457	Yes	11.26	Pass
Boron (Total)	µg/L	0.010	20	1.43	1.42	Yes	0.47	Pass
Cadmium (Total)	mg/L	0.0000050	20	0.00942	0.00714	Yes	17.55	Pass
Cesium (Total)	mg/L	0.000010	20	0.0488	0.0423	Yes	9.29	Pass
Chromium (Total)	mg/L	0.00050	20	0.889	0.706	Yes	14.73	Pass
Cobalt (Total)	mg/L	0.00010	20	0.240	0.181	Yes	17.85	Pass
Copper (Total)	mg/L	0.00050	20	0.747	0.584	Yes	15.69	Pass
Iron (Total)	mg/L	0.010	20	583	473	Yes	13.42	Pass
Lead (Total)	mg/L	0.000050	20	0.276	0.235	Yes	10.42	Pass
Lithium (Total)	mg/L	0.0010	20	0.775	0.660	Yes	10.41	Pass
Manganese (Total)	mg/L	0.00010	20	17.8	12.2	Yes	23.43	Pass
Molybdenum (Total)	mg/L	0.000050	20	0.0291	0.0283	Yes	1.85	Pass
Nickel (Total)	mg/L	0.00050	20	0.714	0.544	Yes	17.24	Pass
Rubidium (Total)	mg/L	0.00020	20	0.666	0.552	Yes	12.10	Pass
Selenium (Total)	mg/L	0.000050	20	0.00749	0.00631	Yes	11.09	Pass
Silicon (Total)	mg/L	0.10	20	468	451	Yes	2.45	Pass
Silver (Total)	mg/L	0.000010	20	0.00288	0.00234	Yes	13.33	Pass
Strontium (Total)	mg/L	0.00020	20	6.92	5.48	Yes	14.91	Pass
Sulfur (Total)	mg/L	0.50	20	308	293	Yes	3.30	Pass
Tellurium (Total)	mg/L	0.00020	20	<0.00200	<0.00200	No	NC	Pass
Thallium (Total)	mg/L	0.000010	20	0.00617	0.00522	Yes	10.82	Pass
Thorium (Total)	mg/L	0.00010	20	0.126	0.106	Yes	11.17	Pass
Tin (Total)	mg/L	0.00010	20	0.0268	0.0316	Yes	11.27	Pass
Titanium (Total)	mg/L	0.00030	20	2.96	2.88	Yes	1.82	Pass
Tungsten (Total)	mg/L	0.00010	20	0.00803	0.00895	Yes	7.36	Pass
Uranium (Total)	mg/L	0.000010	20	0.0511	0.0400	Yes	15.61	Pass
Vanadium (Total)	mg/L	0.00050	20	0.684	0.627	Yes	5.71	Pass
Zinc (Total)	mg/L	0.0030	20	2.68	1.98	Yes	19.07	Pass
Zirconium (Total)	mg/L	0.00020	20	0.0111	0.0109	Yes	1.21	Pass
Aluminum (Dissolved)	mg/L	0.0010	20	0.0016	0.0014	No	NC	Pass
Antimony (Dissolved)	mg/L	0.00010	20	0.00011	0.00012	No	NC	Pass
Arsenic (Dissolved)	mg/L	0.00010	20	0.00076	0.00060	Yes	15.09	Pass
Barium (Dissolved)	mg/L	0.00010	20	0.0321	0.0344	Yes	4.67	Pass
Beryllium (Dissolved)	mg/L	0.000020	20	<0.000020	<0.000020	No	NC	Pass
Bismuth (Dissolved)	mg/L	0.000050	20	<0.000050	<0.000050	No	NC	Pass
Boron (Dissolved)	mg/L	0.010	20	0.792	0.824	Yes	2.66	Pass
Cadmium (Dissolved)	mg/L	0.0000050	20	0.0000111	0.0000111	Yes	0.00	Pass
Cesium (Dissolved)	mg/L	0.000010	20	0.000070	0.000070	Yes	0.00	Pass
Chromium (Dissolved)	mg/L	0.00050	20	<0.00050	<0.00050	No	NC	Pass
Cobalt (Dissolved)	mg/L	0.00010	20	0.00097	0.00090	Yes	4.93	Pass
Iron (Dissolved)	mg/L	0.010	20	<0.010	<0.010	No	NC	Pass
Lead (Dissolved)	mg/L	0.000050	20	<0.000050	<0.000050	No	NC	Pass
Lithium (Dissolved)	mg/L	0.0010	20	0.266	0.258	Yes	2.03	Pass
Manganese (Dissolved)	mg/L	0.00010	20	0.0941	0.0938	Yes	0.21	Pass
Molybdenum (Dissolved)	mg/L	0.000050	20	0.00386	0.00445	Yes	9.70	Pass
Nickel (Dissolved)	mg/L	0.00050	20	0.00377	0.00354	Yes	4.15	Pass
Rubidium (Dissolved)	mg/L	0.00020	20	0.0131	0.0138	Yes	3.50	Pass
Selenium (Dissolved)	mg/L	0.000050	20	<0.000050	<0.000050	No	NC	Pass
Silicon (Dissolved)	mg/L	0.050	20	6.31	6.30	Yes	0.11	Pass
Silver (Dissolved)	mg/L	0.000010	20	<0.000010	<0.000010	No	NC	Pass
Strontium (Dissolved)	mg/L	0.00020	20	3.02	3.06	Yes	0.88	Pass
Sulfur (Dissolved)	mg/L	0.50	20	297	311	Yes	3.09	Pass
Tellurium (Dissolved)	mg/L	0.00020	20	<0.00020	<0.00020	No	NC	Pass
Thallium (Dissolved)	mg/L	0.000010	20	0.000022	0.000036	No	NC	Pass
Thorium (Dissolved)	mg/L	0.00010	20	<0.00010	<0.00010	No	NC	Pass
Tin (Dissolved)	mg/L	0.00010	20	0.00070	0.00077	Yes	6.45	Pass
Titanium (Dissolved)	mg/L	0.00030	20	<0.00030	<0.00030	No	NC	Pass
Tungsten (Dissolved)	mg/L	0.00010	20	0.00072	0.00098	Yes	21.49	Pass
Uranium (Dissolved)	mg/L	0.000010	20	0.00243	0.00259	Yes	4.30	Pass
Vanadium (Dissolved)	mg/L	0.00050	20	<0.00050	<0.00050	No	NC	Pass
Zinc (Dissolved)	mg/L	0.0010	20	0.0021	0.0015	No	NC	Pass
Zirconium (Dissolved)	mg/L	0.00030	20	<0.00030	<0.00030	No	NC	Pass
Ammonia, total (as N)	mg/L	0.0050	25	0.882	0.882	Yes	0.00	Pass
Nitrate (as N)	mg/L	0.020	10	<0.400	<0.400	No	NC	Pass
Nitrite (as N)	mg/L	0.010	10	<0.200	<0.200	No	NC	Pass
Total Nitrogen	mg/L	0.5	10	1.182	1.182	No	NC	Pass
Calcium (Dissolved)	mg/L	0.050	20	239	245	Yes	1.66	Pass
Calcium (Total)	mg/L	0.050	20	4730	4180	Yes	8.06	Pass
Magnesium (Dissolved)	mg/L	0.0050	20	151	144	Yes	3.14	Pass
Magnesium (Total)	mg/L	0.0050	20	2400	2260	Yes	3.97	Pass
Phosphorus (Dissolved)	mg/L	0.050	20	<0.050	<0.050	No	NC	Pass
Phosphorus (Total)	mg/L	0.050	20	14.2	12.6	Yes	7.80	Pass
Potassium (Dissolved)	mg/L	0.050	20	33.6	33.9	Yes	0.59	Pass
Potassium (Total)	mg/L	0.050	20	114	103	Yes	6.65	Pass
Sodium (Dissolved)	mg/L	0.050	20	935	921	Yes	1.00	Pass
Sodium (Total)	mg/L	0.050	20	959	870	Yes	6.38	Pass
Acenaphthene	µg/L	0.010	50	<0.019	<0.010	No	NC	Pass
Acenaphthylene	µg/L	0.010	50	<0.010	<0.010	No	NC	Pass
Acridine	µg/L	0.010	50	<0.091	<0.039	No	NC	Pass
Anthracene	µg/L	0.010	50	<0.020	<0.010	No	NC	Pass
Benz(a)anthracene	µg/L	0.010	50	<0.010	<0.010	No	NC	Pass
Benzo(a)pyrene	µg/L	0.0050	50	<0.0050	<0.0050	No	NC	Pass
Benzo(b+j)fluoranthene	µg/L	0.010	50	<0.015	<0.010	No	NC	Pass
Benzo(b+j+k)fluoranthene	µg/L	0.015	50	<0.021	<0.015	No	NC	Pass
Benzo(g,h,i)perylene	µg/L	0.010	50	<0.010	<0.010	No	NC	Pass
Benzo(k)fluoranthene	µg/L	0.010	50	<0.015	<0.010	No	NC	Pass
Chrysene	µg/L	0.010	50	<0.014	<0.010	No	NC	Pass
Dibenz(a,h)anthracene	µg/L	0.0050	50	<0.0050	<0.0050	No	NC	Pass
Fluoranthene	µg/L	0.010	50	<0.056	0.026	No	NC	Pass
Fluorene	µg/L	0.010	50	0.035	0.016	No	NC	Pass
Indeno(1,2,3-c,d)pyrene	µg/L	0.010	50	<0.010	<0.010	No	NC	Pass
Methylnaphthalene, 1+2-	µg/L	0.015	50	0.127	0.063	No	NC	Pass
Methylnaphthalene, 1-	µg/L	0.010	50	0.050	0.025	No	NC	Pass
Methylnaphthalene, 2-	µg/L	0.010	50	0.077	0.038	No	NC	Pass
Naphthalene	µg/L	0.050	50	0.059	<0.050	No	NC	Pass
Phenanthrene	µg/L	0.020	50	0.099	0.040	No	NC	Pass
Pyrene	µg/L	0.010	50	0.095	0.050	Yes	37.50	Pass
Quinoline	µg/L	0.050	50	<0.050	<0.050	No	NC	Pass
B(a)P total potency equiva	µg/L	0.010	50	<0.010	<0.010	No	NC	Pass
PAHs, high molecular we	µg/L	0.030	50	0.095	0.076	No	14.29	Pass
PAHs, low molecular wei	µg/L	0.060	50	0.193	<0.060	No	NC	Pass
PAHs, total (CCME sewe	µg/L	0.070	50	0.415	0.195	No	42.93	Pass
PAHs, total (EPA 16)	µg/L	0.065	50	0.288	0.132	No	44.07	Pass
Benzene	mg/L	0.00050	30	<0.00050	<0.00050	No	NC	Pass
Ethylbenzene	mg/L	0.00050	30	<0.00050	<0.00050	No	NC	Pass
Toluene	mg/L	0.00050	30	<0.00050	<0.00050	No	NC	Pass
Xylene, m+p-	mg/L	0.00040	30	<0.00040	<0.00040	No	NC	Pass
Xylene, o-	mg/L	0.00030	30	<0.00030	<0.00030	No	NC	Pass
Xylenes, total	mg/L	0.00050	30	<0.00050	<0.00050	No	NC	Pass
Styrene	mg/L	0.00050	30	<0.00050	<0.00050	No	NC	Pass
PHC F1 (C6-C10) minus	mg/L	0.10	30	<0.10	<0.10	No	NC	Pass
PHC F2 (>C10-C16)	mg/L	0.10	30	<0.10	<0.10	No	NC	Pass
PHC F2 (>C10-C16)	mg/L	0.10	30	<0.10	<0.10	No	NC	Pass

Notes
NC = Not calculated
mbgs = meters below ground surface
RPD = relative percent difference
mg/L = milligrams per litre
µg/L = micrograms per litre

CERTIFICATE OF ANALYSIS (GUIDELINE EVALUATION)

Work Order	: **WP2501636**	Page	: 1 of 17
Client	: **AECOM Canada ULC**	Laboratory	: ALS Environmental - Winnipeg
Contact	: Manny Papadimitropoulos	Account Manager	: Judy Dalmaijer
Address	: 99 Commerce Drive Winnipeg MB Canada R3P 0Y7	Address	: 1329 Niakwa Road East, Unit 12 Winnipeg, Manitoba Canada R2J 3T4
Telephone	: 204 477 5381	Telephone	: +1 204 255 9720
Project	: 60728226	Date Samples Received	: 06-Feb-2025 16:33
PO	: 1687450	Date Analysis Commenced	: 07-Feb-2025
C-O-C number	: ----	Issue Date	: 11-Feb-2025 17:25
Sampler	: ----		
Site	: ----		
Quote number	: 2024 Standing offer		
No. of samples received	: 3		
No. of samples analysed	: 3		

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results
- Guideline Comparison

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QC Interpretive report to assist with Quality Review and Sample Receipt Notification (SRN).

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is conducted in accordance with US FDA 21 CFR Part 11.

Signatories	*Position*	*Laboratory Department*
Jeremy Gingras	Supervisor - Semi-Volatile Instrumentation	Organics, Waterloo, Ontario
Kevin Baxter		Inorganics, Winnipeg, Manitoba
Kevin Baxter		Metals, Winnipeg, Manitoba
Leila Conyard	Lab Assistant	Metals, Winnipeg, Manitoba
Michelle Michalchuk	Analyst	Organics, Winnipeg, Manitoba
Ryan Velasco		Organics, Winnipeg, Manitoba

General Comments

The analytical methods used by ALS are developed using internationally recognized reference methods (where available), such as those published by US EPA, APHA Standard Methods, ASTM, ISO, Environment Canada, BC MOE, and Ontario MOE. Refer to the ALS Quality Control Interpretive report (QCI) for applicable references and methodology summaries. Reference methods may incorporate modifications to improve performance.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.
Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QA/QC Compliance Assessment to assist with Quality Review and Sample Receipt Notification.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to fitness for a particular purpose, or non -infringement. ALS assumes no responsibility for errors or omissions in the information. Guidelines are not adjusted for the hardness, pH or temperature of the sample (the most conservative values are used). Measurement uncertainty is not applied to test results prior to comparison with specified criteria values.

Key : LOR: Limit of Reporting (detection limit).

Unit	Description
	>: greater than.
	<: less than.
	Red shading is applied where the result or the LOR is greater than the Guideline Upper Limit (or lower than the Guideline Lower Limit, if applicable).
	For drinking water samples, Red shading is applied where the result for E.coli, fecal or total coliforms is greater than or equal to the Guideline Upper Limit .

Qualifiers

Qualifier	Description
DLM	*Detection Limit Adjusted due to sample matrix effects (e.g. chemical interference, colour, turbidity).*
RRR	*Refer to report comments for issues regarding this analysis.*

Analytical Results

Sub-Matrix: **Water**
(Matrix: **Water**)

			Client sample ID	**TH24-01**						
			Sampling date/time	06-Feb-2025 11:00						
Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-001	--	--	--	--	--	--
Anions and Nutrients										
Ammonia, total (as N)	E298/WP	0.0050	mg/L	0.891	--	--	--	--	--	--
Nitrate (as N)	E235.NO3/WP	0.020	mg/L	<1.00 DLM	--	--	--	--	--	--
Nitrite (as N)	E235.NO2/WP	0.010	mg/L	<0.500 DLM	--	--	--	--	--	--
Total Metals										
Aluminum, total	E420/WP	0.0030	mg/L	108	--	--	--	--	--	--
Antimony, total	E420/WP	0.00010	mg/L	0.00198	--	--	--	--	--	--
Arsenic, total	E420/WP	0.00010	mg/L	0.0778	--	--	--	--	--	--
Barium, total	E420/WP	0.00010	mg/L	1.31	--	--	--	--	--	--
Beryllium, total	E420/WP	0.000020	mg/L	0.00623	--	--	--	--	--	--
Bismuth, total	E420/WP	0.000050	mg/L	0.00179	--	--	--	--	--	--
Boron, total	E420/WP	0.010	mg/L	1.18	--	--	--	--	--	--
Cadmium, total	E420/WP	0.0000050	mg/L	0.00228	--	--	--	--	--	--
Calcium, total	E420/WP	0.050	mg/L	1070	--	--	--	--	--	--
Cesium, total	E420/WP	0.000010	mg/L	0.0201	--	--	--	--	--	--
Chromium, total	E420/WP	0.00050	mg/L	0.234	--	--	--	--	--	--
Cobalt, total	E420/WP	0.00010	mg/L	0.0766	--	--	--	--	--	--
Copper, total	E420/WP	0.00050	mg/L	0.219	--	--	--	--	--	--
Iron, total	E420/WP	0.010	mg/L	187	--	--	--	--	--	--
Lead, total	E420/WP	0.000050	mg/L	0.0944	--	--	--	--	--	--
Lithium, total	E420/WP	0.0010	mg/L	0.469	--	--	--	--	--	--
Magnesium, total	E420/WP	0.0050	mg/L	587	--	--	--	--	--	--
Manganese, total	E420/WP	0.00010	mg/L	3.00	--	--	--	--	--	--
Molybdenum, total	E420/WP	0.000050	mg/L	0.0142	--	--	--	--	--	--
Nickel, total	E420/WP	0.00050	mg/L	0.228	--	--	--	--	--	--
Phosphorus, total	E420/WP	0.050	mg/L	3.96	--	--	--	--	--	--
Potassium, total	E420/WP	0.050	mg/L	73.1	--	--	--	--	--	--
Rubidium, total	E420/WP	0.00020	mg/L	0.273	--	--	--	--	--	--
Selenium, total	E420/WP	0.000050	mg/L	0.00215	--	--	--	--	--	--
Silicon, total	E420/WP	0.10	mg/L	236	--	--	--	--	--	--
Silver, total	E420/WP	0.000010	mg/L	0.000698	--	--	--	--	--	--
Sodium, total	E420/WP	0.050	mg/L	1090	--	--	--	--	--	--
Strontium, total	E420/WP	0.00020	mg/L	4.38	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-001 (Continued)	--	--	--	--	--	--
Total Metals - Continued										
Sulfur, total	E420/WP	0.50	mg/L	342	--	--	--	--	--	--
Tellurium, total	E420/WP	0.00020	mg/L	<0.00200 DLM	--	--	--	--	--	--
Thallium, total	E420/WP	0.000010	mg/L	0.00203	--	--	--	--	--	--
Thorium, total	E420/WP	0.00010	mg/L	0.0435	--	--	--	--	--	--
Tin, total	E420/WP	0.00010	mg/L	0.0583	--	--	--	--	--	--
Titanium, total	E420/WP	0.00030	mg/L	2.05	--	--	--	--	--	--
Tungsten, total	E420/WP	0.00010	mg/L	0.00288	--	--	--	--	--	--
Uranium, total	E420/WP	0.000010	mg/L	0.0137	--	--	--	--	--	--
Vanadium, total	E420/WP	0.00050	mg/L	0.310	--	--	--	--	--	--
Zinc, total	E420/WP	0.0030	mg/L	0.756	--	--	--	--	--	--
Zirconium, total	E420/WP	0.00020	mg/L	0.0150	--	--	--	--	--	--
Dissolved Metals										
Aluminum, dissolved	E421/WP	0.0010	mg/L	0.0015	--	--	--	--	--	--
Antimony, dissolved	E421/WP	0.00010	mg/L	<0.00010	--	--	--	--	--	--
Arsenic, dissolved	E421/WP	0.00010	mg/L	0.00087	--	--	--	--	--	--
Barium, dissolved	E421/WP	0.00010	mg/L	0.0217	--	--	--	--	--	--
Beryllium, dissolved	E421/WP	0.000020	mg/L	Not Detected	--	--	--	--	--	--
Bismuth, dissolved	E421/WP	0.000050	mg/L	Not Detected	--	--	--	--	--	--
Boron, dissolved	E421/WP	0.010	mg/L	0.809	--	--	--	--	--	--
Cadmium, dissolved	E421/WP	0.0000050	mg/L	<0.0000050	--	--	--	--	--	--
Calcium, dissolved	E421/WP	0.050	mg/L	257	--	--	--	--	--	--
Cesium, dissolved	E421/WP	0.000010	mg/L	0.000050	--	--	--	--	--	--
Chromium, dissolved	E421/WP	0.00050	mg/L	Not Detected	--	--	--	--	--	--
Cobalt, dissolved	E421/WP	0.00010	mg/L	0.00137	--	--	--	--	--	--
Copper, dissolved	E421/WP	0.00020	mg/L	<0.00020	--	--	--	--	--	--
Iron, dissolved	E421/WP	0.010	mg/L	<0.010	--	--	--	--	--	--
Lead, dissolved	E421/WP	0.000050	mg/L	Not Detected	--	--	--	--	--	--
Lithium, dissolved	E421/WP	0.0010	mg/L	0.262	--	--	--	--	--	--
Magnesium, dissolved	E421/WP	0.0050	mg/L	137	--	--	--	--	--	--
Manganese, dissolved	E421/WP	0.00010	mg/L	0.103	--	--	--	--	--	--
Molybdenum, dissolved	E421/WP	0.000050	mg/L	0.00743	--	--	--	--	--	--
Nickel, dissolved	E421/WP	0.00050	mg/L	0.00401	--	--	--	--	--	--
Phosphorus, dissolved	E421/WP	0.050	mg/L	Not Detected	--	--	--	--	--	--
Potassium, dissolved	E421/WP	0.050	mg/L	41.9	--	--	--	--	--	--
Rubidium, dissolved	E421/WP	0.00020	mg/L	0.0214	--	--	--	--	--	--
Selenium, dissolved	E421/WP	0.000050	mg/L	<0.000050	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-001 (Continued)	--	--	--	--	--	--
Dissolved Metals - Continued										
Silicon, dissolved	E421/WP	0.050	mg/L	4.24	--	--	--	--	--	--
Silver, dissolved	E421/WP	0.000010	mg/L	<0.000010	--	--	--	--	--	--
Sodium, dissolved	E421/WP	0.050	mg/L	1110	--	--	--	--	--	--
Strontium, dissolved	E421/WP	0.00020	mg/L	3.53	--	--	--	--	--	--
Sulfur, dissolved	E421/WP	0.50	mg/L	347	--	--	--	--	--	--
Tellurium, dissolved	E421/WP	0.00020	mg/L	Not Detected	--	--	--	--	--	--
Thallium, dissolved	E421/WP	0.000010	mg/L	0.000010	--	--	--	--	--	--
Thorium, dissolved	E421/WP	0.00010	mg/L	Not Detected	--	--	--	--	--	--
Tin, dissolved	E421/WP	0.00010	mg/L	0.00194	--	--	--	--	--	--
Titanium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Tungsten, dissolved	E421/WP	0.00010	mg/L	0.00042	--	--	--	--	--	--
Uranium, dissolved	E421/WP	0.000010	mg/L	0.00247	--	--	--	--	--	--
Vanadium, dissolved	E421/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Zinc, dissolved	E421/WP	0.0010	mg/L	0.0015	--	--	--	--	--	--
Zirconium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Dissolved metals filtration location	EP421/WP		-	Laboratory	--	--	--	--	--	--
Volatile Organic Compounds										
Benzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Ethylbenzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Styrene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Toluene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Xylene, m+p-	E611A/WP	0.00040	mg/L	<0.00040	--	--	--	--	--	--
Xylene, o-	E611A/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Xylenes, total	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Hydrocarbons										
F1 (C6-C10)	E581.F1/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F2 (C10-C16)	E601/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F1-BTEX	EC580/WP	0.100	mg/L	<0.100	--	--	--	--	--	--
Bromobenzotrifluoride, 2- (F2-F4 surrogate)	E601/WP	1.0	%	76.4	--	--	--	--	--	--
Dichlorotoluene, 3,4-	E581.F1/WP	1.0	%	92.7	--	--	--	--	--	--
Bromofluorobenzene, 4-	E611A/WP	1.0	%	101	--	--	--	--	--	--
Difluorobenzene, 1,4-	E611A/WP	1.0	%	102	--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons										
Acenaphthene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-001 (Continued)	--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons - Continued										
Acenaphthylene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Acridine	E641A/WT	0.010	µg/L	<0.016 DLM RRR	--	--	--	--	--	--
Anthracene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Benz(a)anthracene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Benzo(a)pyrene	E641A/WT	0.0050	µg/L	<0.0050	--	--	--	--	--	--
Benzo(b+j)fluoranthene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Benzo(b+j+k)fluoranthene	E641A/WT	0.015	µg/L	<0.015	--	--	--	--	--	--
Benzo(g,h,i)perylene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Benzo(k)fluoranthene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Chrysene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Dibenz(a,h)anthracene	E641A/WT	0.0050	µg/L	<0.0050	--	--	--	--	--	--
Fluoranthene	E641A/WT	0.010	µg/L	0.015	--	--	--	--	--	--
Fluorene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Indeno(1,2,3-c,d)pyrene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
Methylnaphthalene, 1+2-	E641A/WT	0.015	µg/L	0.059	--	--	--	--	--	--
Methylnaphthalene, 1-	E641A/WT	0.010	µg/L	0.024	--	--	--	--	--	--
Methylnaphthalene, 2-	E641A/WT	0.010	µg/L	0.035	--	--	--	--	--	--
Naphthalene	E641A/WT	0.050	µg/L	<0.050	--	--	--	--	--	--
Phenanthrene	E641A/WT	0.020	µg/L	0.025	--	--	--	--	--	--
Pyrene	E641A/WT	0.010	µg/L	0.030	--	--	--	--	--	--
Quinoline	E641A/WT	0.050	µg/L	<0.050	--	--	--	--	--	--
B(a)P total potency equivalents [B(a)P TPE]	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--
PAHs, high molecular weight (BC AWQ)	E641A/WT	0.030	µg/L	0.045	--	--	--	--	--	--
PAHs, low molecular weight (BC AWQ)	E641A/WT	0.060	µg/L	<0.060	--	--	--	--	--	--
PAHs, total (CCME sewer 18)	E641A/WT	0.070	µg/L	0.129	--	--	--	--	--	--
PAHs, total (EPA 16)	E641A/WT	0.065	µg/L	0.070	--	--	--	--	--	--
Chrysene-d12	E641A/WT	0.1	%	93.7	--	--	--	--	--	--
Naphthalene-d8	E641A/WT	0.1	%	103	--	--	--	--	--	--
Phenanthrene-d10	E641A/WT	0.1	%	108	--	--	--	--	--	--

Please refer to the General Comments section for an explanation of any result qualifiers detected.

Please refer to the Accreditation section for an explanation of analyte accreditations.

No Breaches Found

Key:

Analytical Results

Sub-Matrix: **Water**
(Matrix: **Water)**

			Client sample ID	**TH24-05**						
			Sampling date/time	06-Feb-2025 12:00						
Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-002	--	--	--	--	--	--
Anions and Nutrients										
Ammonia, total (as N)	E298/WP	0.0050	mg/L	0.882	--	--	--	--	--	--
Nitrate (as N)	E235.NO3/WP	0.020	mg/L	<0.400 DLM	--	--	--	--	--	--
Nitrite (as N)	E235.NO2/WP	0.010	mg/L	<0.200 DLM	--	--	--	--	--	--
Total Metals										
Aluminum, total	E420/WP	0.0030	mg/L	219	--	--	--	--	--	--
Antimony, total	E420/WP	0.00010	mg/L	0.00293	--	--	--	--	--	--
Arsenic, total	E420/WP	0.00010	mg/L	0.171	--	--	--	--	--	--
Barium, total	E420/WP	0.00010	mg/L	3.64	--	--	--	--	--	--
Beryllium, total	E420/WP	0.000020	mg/L	0.0173	--	--	--	--	--	--
Bismuth, total	E420/WP	0.000050	mg/L	0.00544	--	--	--	--	--	--
Boron, total	E420/WP	0.010	mg/L	1.43	--	--	--	--	--	--
Cadmium, total	E420/WP	0.0000050	mg/L	0.00942	--	--	--	--	--	--
Calcium, total	E420/WP	0.050	mg/L	4730	--	--	--	--	--	--
Cesium, total	E420/WP	0.000010	mg/L	0.0488	--	--	--	--	--	--
Chromium, total	E420/WP	0.00050	mg/L	0.889	--	--	--	--	--	--
Cobalt, total	E420/WP	0.00010	mg/L	0.240	--	--	--	--	--	--
Copper, total	E420/WP	0.00050	mg/L	0.747	--	--	--	--	--	--
Iron, total	E420/WP	0.010	mg/L	583	--	--	--	--	--	--
Lead, total	E420/WP	0.000050	mg/L	0.276	--	--	--	--	--	--
Lithium, total	E420/WP	0.0010	mg/L	0.775	--	--	--	--	--	--
Magnesium, total	E420/WP	0.0050	mg/L	2400	--	--	--	--	--	--
Manganese, total	E420/WP	0.00010	mg/L	17.8	--	--	--	--	--	--
Molybdenum, total	E420/WP	0.000050	mg/L	0.0291	--	--	--	--	--	--
Nickel, total	E420/WP	0.00050	mg/L	0.714	--	--	--	--	--	--
Phosphorus, total	E420/WP	0.050	mg/L	14.2	--	--	--	--	--	--
Potassium, total	E420/WP	0.050	mg/L	114	--	--	--	--	--	--
Rubidium, total	E420/WP	0.00020	mg/L	0.666	--	--	--	--	--	--
Selenium, total	E420/WP	0.000050	mg/L	0.00749	--	--	--	--	--	--
Silicon, total	E420/WP	0.10	mg/L	468	--	--	--	--	--	--
Silver, total	E420/WP	0.000010	mg/L	0.00288	--	--	--	--	--	--
Sodium, total	E420/WP	0.050	mg/L	959	--	--	--	--	--	--
Strontium, total	E420/WP	0.00020	mg/L	6.92	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-002 (Continued)	--	--	--	--	--	--
Total Metals - Continued										
Sulfur, total	E420/WP	0.50	mg/L	308	--	--	--	--	--	--
Tellurium, total	E420/WP	0.00020	mg/L	<0.00200 DLM	--	--	--	--	--	--
Thallium, total	E420/WP	0.000010	mg/L	0.00617	--	--	--	--	--	--
Thorium, total	E420/WP	0.00010	mg/L	0.126	--	--	--	--	--	--
Tin, total	E420/WP	0.00010	mg/L	0.0268	--	--	--	--	--	--
Titanium, total	E420/WP	0.00030	mg/L	2.96	--	--	--	--	--	--
Tungsten, total	E420/WP	0.00010	mg/L	0.00803	--	--	--	--	--	--
Uranium, total	E420/WP	0.000010	mg/L	0.0511	--	--	--	--	--	--
Vanadium, total	E420/WP	0.00050	mg/L	0.684	--	--	--	--	--	--
Zinc, total	E420/WP	0.0030	mg/L	2.68	--	--	--	--	--	--
Zirconium, total	E420/WP	0.00020	mg/L	0.0111	--	--	--	--	--	--
Dissolved Metals										
Aluminum, dissolved	E421/WP	0.0010	mg/L	0.0016	--	--	--	--	--	--
Antimony, dissolved	E421/WP	0.00010	mg/L	0.00011	--	--	--	--	--	--
Arsenic, dissolved	E421/WP	0.00010	mg/L	0.00076	--	--	--	--	--	--
Barium, dissolved	E421/WP	0.00010	mg/L	0.0321	--	--	--	--	--	--
Beryllium, dissolved	E421/WP	0.000020	mg/L	Not Detected	--	--	--	--	--	--
Bismuth, dissolved	E421/WP	0.000050	mg/L	Not Detected	--	--	--	--	--	--
Boron, dissolved	E421/WP	0.010	mg/L	0.792	--	--	--	--	--	--
Cadmium, dissolved	E421/WP	0.0000050	mg/L	0.0000111	--	--	--	--	--	--
Calcium, dissolved	E421/WP	0.050	mg/L	239	--	--	--	--	--	--
Cesium, dissolved	E421/WP	0.000010	mg/L	0.000070	--	--	--	--	--	--
Chromium, dissolved	E421/WP	0.00050	mg/L	Not Detected	--	--	--	--	--	--
Cobalt, dissolved	E421/WP	0.00010	mg/L	0.00097	--	--	--	--	--	--
Copper, dissolved	E421/WP	0.00020	mg/L	<0.00020	--	--	--	--	--	--
Iron, dissolved	E421/WP	0.010	mg/L	<0.010	--	--	--	--	--	--
Lead, dissolved	E421/WP	0.000050	mg/L	Not Detected	--	--	--	--	--	--
Lithium, dissolved	E421/WP	0.0010	mg/L	0.266	--	--	--	--	--	--
Magnesium, dissolved	E421/WP	0.0050	mg/L	151	--	--	--	--	--	--
Manganese, dissolved	E421/WP	0.00010	mg/L	0.0941	--	--	--	--	--	--
Molybdenum, dissolved	E421/WP	0.000050	mg/L	0.00386	--	--	--	--	--	--
Nickel, dissolved	E421/WP	0.00050	mg/L	0.00377	--	--	--	--	--	--
Phosphorus, dissolved	E421/WP	0.050	mg/L	<0.050	--	--	--	--	--	--
Potassium, dissolved	E421/WP	0.050	mg/L	33.6	--	--	--	--	--	--
Rubidium, dissolved	E421/WP	0.00020	mg/L	0.0131	--	--	--	--	--	--
Selenium, dissolved	E421/WP	0.000050	mg/L	<0.000050	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-002 (Continued)	--	--	--	--	--	--
Dissolved Metals - Continued										
Silicon, dissolved	E421/WP	0.050	mg/L	6.31	--	--	--	--	--	--
Silver, dissolved	E421/WP	0.000010	mg/L	Not Detected	--	--	--	--	--	--
Sodium, dissolved	E421/WP	0.050	mg/L	935	--	--	--	--	--	--
Strontium, dissolved	E421/WP	0.00020	mg/L	3.02	--	--	--	--	--	--
Sulfur, dissolved	E421/WP	0.50	mg/L	297	--	--	--	--	--	--
Tellurium, dissolved	E421/WP	0.00020	mg/L	Not Detected	--	--	--	--	--	--
Thallium, dissolved	E421/WP	0.000010	mg/L	0.000022	--	--	--	--	--	--
Thorium, dissolved	E421/WP	0.00010	mg/L	Not Detected	--	--	--	--	--	--
Tin, dissolved	E421/WP	0.00010	mg/L	0.00070	--	--	--	--	--	--
Titanium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Tungsten, dissolved	E421/WP	0.00010	mg/L	0.00072	--	--	--	--	--	--
Uranium, dissolved	E421/WP	0.000010	mg/L	0.00243	--	--	--	--	--	--
Vanadium, dissolved	E421/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Zinc, dissolved	E421/WP	0.0010	mg/L	0.0021	--	--	--	--	--	--
Zirconium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Dissolved metals filtration location	EP421/WP		-	Laboratory	--	--	--	--	--	--
Volatile Organic Compounds										
Benzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Ethylbenzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Styrene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Toluene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Xylene, m+p-	E611A/WP	0.00040	mg/L	<0.00040	--	--	--	--	--	--
Xylene, o-	E611A/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Xylenes, total	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Hydrocarbons										
F1 (C6-C10)	E581.F1/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F2 (C10-C16)	E601/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F1-BTEX	EC580/WP	0.100	mg/L	<0.100	--	--	--	--	--	--
Bromobenzotrifluoride, 2- (F2-F4 surrogate)	E601/WP	1.0	%	81.8	--	--	--	--	--	--
Dichlorotoluene, 3,4-	E581.F1/WP	1.0	%	89.5	--	--	--	--	--	--
Bromofluorobenzene, 4-	E611A/WP	1.0	%	95.0	--	--	--	--	--	--
Difluorobenzene, 1,4-	E611A/WP	1.0	%	105	--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons										
Acenaphthene	E641A/WT	0.010	µg/L	<0.019 DLM	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-002 (Continued)		--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons - Continued											
Acenaphthylene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Acridine	E641A/WT	0.010	µg/L	<0.091	DLM RRR	--	--	--	--	--	--
Anthracene	E641A/WT	0.010	µg/L	<0.020	DLM	--	--	--	--	--	--
Benz(a)anthracene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benzo(a)pyrene	E641A/WT	0.0050	µg/L	<0.0050		--	--	--	--	--	--
Benzo(b+j)fluoranthene	E641A/WT	0.010	µg/L	<0.015	DLM	--	--	--	--	--	--
Benzo(b+j+k)fluoranthene	E641A/WT	0.015	µg/L	<0.021		--	--	--	--	--	--
Benzo(g,h,i)perylene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benzo(k)fluoranthene	E641A/WT	0.010	µg/L	<0.015	DLM	--	--	--	--	--	--
Chrysene	E641A/WT	0.010	µg/L	<0.014	DLM	--	--	--	--	--	--
Dibenz(a,h)anthracene	E641A/WT	0.0050	µg/L	<0.0050		--	--	--	--	--	--
Fluoranthene	E641A/WT	0.010	µg/L	<0.056	DLM	--	--	--	--	--	--
Fluorene	E641A/WT	0.010	µg/L	0.035		--	--	--	--	--	--
Indeno(1,2,3-c,d)pyrene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Methylnaphthalene, 1+2-	E641A/WT	0.015	µg/L	0.127		--	--	--	--	--	--
Methylnaphthalene, 1-	E641A/WT	0.010	µg/L	0.050		--	--	--	--	--	--
Methylnaphthalene, 2-	E641A/WT	0.010	µg/L	0.077		--	--	--	--	--	--
Naphthalene	E641A/WT	0.050	µg/L	0.059		--	--	--	--	--	--
Phenanthrene	E641A/WT	0.020	µg/L	0.099		--	--	--	--	--	--
Pyrene	E641A/WT	0.010	µg/L	0.095		--	--	--	--	--	--
Quinoline	E641A/WT	0.050	µg/L	<0.050		--	--	--	--	--	--
B(a)P total potency equivalents [B(a)P TPE]	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
PAHs, high molecular weight (BC AWQ)	E641A/WT	0.030	µg/L	0.095		--	--	--	--	--	--
PAHs, low molecular weight (BC AWQ)	E641A/WT	0.060	µg/L	0.193		--	--	--	--	--	--
PAHs, total (CCME sewer 18)	E641A/WT	0.070	µg/L	0.415		--	--	--	--	--	--
PAHs, total (EPA 16)	E641A/WT	0.065	µg/L	0.288		--	--	--	--	--	--
Chrysene-d12	E641A/WT	0.1	%	96.4		--	--	--	--	--	--
Naphthalene-d8	E641A/WT	0.1	%	103		--	--	--	--	--	--
Phenanthrene-d10	E641A/WT	0.1	%	110		--	--	--	--	--	--

Please refer to the General Comments section for an explanation of any result qualifiers detected.

Please refer to the Accreditation section for an explanation of analyte accreditations.

No Breaches Found

Key:

Analytical Results

			Client sample ID	**DUP-01**						
Sub-Matrix: **Water** (Matrix: **Water**)			*Sampling date/time*	06-Feb-2025 13:00						
Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-003	--	--	--	--	--	--
Anions and Nutrients										
Ammonia, total (as N)	E298/WP	0.0050	mg/L	0.882	--	--	--	--	--	--
Nitrate (as N)	E235.NO3/WP	0.020	mg/L	<0.400 DLM	--	--	--	--	--	--
Nitrite (as N)	E235.NO2/WP	0.010	mg/L	<0.200 DLM	--	--	--	--	--	--
Total Metals										
Aluminum, total	E420/WP	0.0030	mg/L	203	--	--	--	--	--	--
Antimony, total	E420/WP	0.00010	mg/L	0.00299	--	--	--	--	--	--
Arsenic, total	E420/WP	0.00010	mg/L	0.153	--	--	--	--	--	--
Barium, total	E420/WP	0.00010	mg/L	3.23	--	--	--	--	--	--
Beryllium, total	E420/WP	0.000020	mg/L	0.0156	--	--	--	--	--	--
Bismuth, total	E420/WP	0.000050	mg/L	0.00457	--	--	--	--	--	--
Boron, total	E420/WP	0.010	mg/L	1.42	--	--	--	--	--	--
Cadmium, total	E420/WP	0.0000050	mg/L	0.00714	--	--	--	--	--	--
Calcium, total	E420/WP	0.050	mg/L	4180	--	--	--	--	--	--
Cesium, total	E420/WP	0.000010	mg/L	0.0423	--	--	--	--	--	--
Chromium, total	E420/WP	0.00050	mg/L	0.706	--	--	--	--	--	--
Cobalt, total	E420/WP	0.00010	mg/L	0.181	--	--	--	--	--	--
Copper, total	E420/WP	0.00050	mg/L	0.584	--	--	--	--	--	--
Iron, total	E420/WP	0.010	mg/L	473	--	--	--	--	--	--
Lead, total	E420/WP	0.000050	mg/L	0.235	--	--	--	--	--	--
Lithium, total	E420/WP	0.0010	mg/L	0.660	--	--	--	--	--	--
Magnesium, total	E420/WP	0.0050	mg/L	2260	--	--	--	--	--	--
Manganese, total	E420/WP	0.00010	mg/L	12.2	--	--	--	--	--	--
Molybdenum, total	E420/WP	0.000050	mg/L	0.0283	--	--	--	--	--	--
Nickel, total	E420/WP	0.00050	mg/L	0.544	--	--	--	--	--	--
Phosphorus, total	E420/WP	0.050	mg/L	12.6	--	--	--	--	--	--
Potassium, total	E420/WP	0.050	mg/L	103	--	--	--	--	--	--
Rubidium, total	E420/WP	0.00020	mg/L	0.552	--	--	--	--	--	--
Selenium, total	E420/WP	0.000050	mg/L	0.00631	--	--	--	--	--	--
Silicon, total	E420/WP	0.10	mg/L	451	--	--	--	--	--	--
Silver, total	E420/WP	0.000010	mg/L	0.00234	--	--	--	--	--	--
Sodium, total	E420/WP	0.050	mg/L	870	--	--	--	--	--	--
Strontium, total	E420/WP	0.00020	mg/L	5.48	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-003 (Continued)	--	--	--	--	--	--
Total Metals - Continued										
Sulfur, total	E420/WP	0.50	mg/L	293	--	--	--	--	--	--
Tellurium, total	E420/WP	0.00020	mg/L	<0.00200 DLM	--	--	--	--	--	--
Thallium, total	E420/WP	0.000010	mg/L	0.00522	--	--	--	--	--	--
Thorium, total	E420/WP	0.00010	mg/L	0.106	--	--	--	--	--	--
Tin, total	E420/WP	0.00010	mg/L	0.0316	--	--	--	--	--	--
Titanium, total	E420/WP	0.00030	mg/L	2.88	--	--	--	--	--	--
Tungsten, total	E420/WP	0.00010	mg/L	0.00895	--	--	--	--	--	--
Uranium, total	E420/WP	0.000010	mg/L	0.0400	--	--	--	--	--	--
Vanadium, total	E420/WP	0.00050	mg/L	0.627	--	--	--	--	--	--
Zinc, total	E420/WP	0.0030	mg/L	1.98	--	--	--	--	--	--
Zirconium, total	E420/WP	0.00020	mg/L	0.0109	--	--	--	--	--	--
Dissolved Metals										
Aluminum, dissolved	E421/WP	0.0010	mg/L	0.0014	--	--	--	--	--	--
Antimony, dissolved	E421/WP	0.00010	mg/L	0.00012	--	--	--	--	--	--
Arsenic, dissolved	E421/WP	0.00010	mg/L	0.00060	--	--	--	--	--	--
Barium, dissolved	E421/WP	0.00010	mg/L	0.0344	--	--	--	--	--	--
Beryllium, dissolved	E421/WP	0.000020	mg/L	Not Detected	--	--	--	--	--	--
Bismuth, dissolved	E421/WP	0.000050	mg/L	Not Detected	--	--	--	--	--	--
Boron, dissolved	E421/WP	0.010	mg/L	0.824	--	--	--	--	--	--
Cadmium, dissolved	E421/WP	0.0000050	mg/L	0.0000111	--	--	--	--	--	--
Calcium, dissolved	E421/WP	0.050	mg/L	245	--	--	--	--	--	--
Cesium, dissolved	E421/WP	0.000010	mg/L	0.000070	--	--	--	--	--	--
Chromium, dissolved	E421/WP	0.00050	mg/L	Not Detected	--	--	--	--	--	--
Cobalt, dissolved	E421/WP	0.00010	mg/L	0.00090	--	--	--	--	--	--
Copper, dissolved	E421/WP	0.00020	mg/L	<0.00020	--	--	--	--	--	--
Iron, dissolved	E421/WP	0.010	mg/L	<0.010	--	--	--	--	--	--
Lead, dissolved	E421/WP	0.000050	mg/L	<0.000050	--	--	--	--	--	--
Lithium, dissolved	E421/WP	0.0010	mg/L	0.258	--	--	--	--	--	--
Magnesium, dissolved	E421/WP	0.0050	mg/L	144	--	--	--	--	--	--
Manganese, dissolved	E421/WP	0.00010	mg/L	0.0938	--	--	--	--	--	--
Molybdenum, dissolved	E421/WP	0.000050	mg/L	0.00445	--	--	--	--	--	--
Nickel, dissolved	E421/WP	0.00050	mg/L	0.00354	--	--	--	--	--	--
Phosphorus, dissolved	E421/WP	0.050	mg/L	Not Detected	--	--	--	--	--	--
Potassium, dissolved	E421/WP	0.050	mg/L	33.9	--	--	--	--	--	--
Rubidium, dissolved	E421/WP	0.00020	mg/L	0.0138	--	--	--	--	--	--
Selenium, dissolved	E421/WP	0.000050	mg/L	<0.000050	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-003 (Continued)	--	--	--	--	--	--
Dissolved Metals - Continued										
Silicon, dissolved	E421/WP	0.050	mg/L	6.30	--	--	--	--	--	--
Silver, dissolved	E421/WP	0.000010	mg/L	<0.000010	--	--	--	--	--	--
Sodium, dissolved	E421/WP	0.050	mg/L	921	--	--	--	--	--	--
Strontium, dissolved	E421/WP	0.00020	mg/L	3.06	--	--	--	--	--	--
Sulfur, dissolved	E421/WP	0.50	mg/L	311	--	--	--	--	--	--
Tellurium, dissolved	E421/WP	0.00020	mg/L	Not Detected	--	--	--	--	--	--
Thallium, dissolved	E421/WP	0.000010	mg/L	0.000036	--	--	--	--	--	--
Thorium, dissolved	E421/WP	0.00010	mg/L	Not Detected	--	--	--	--	--	--
Tin, dissolved	E421/WP	0.00010	mg/L	0.00077	--	--	--	--	--	--
Titanium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Tungsten, dissolved	E421/WP	0.00010	mg/L	0.00098	--	--	--	--	--	--
Uranium, dissolved	E421/WP	0.000010	mg/L	0.00259	--	--	--	--	--	--
Vanadium, dissolved	E421/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Zinc, dissolved	E421/WP	0.0010	mg/L	0.0015	--	--	--	--	--	--
Zirconium, dissolved	E421/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Dissolved metals filtration location	EP421/WP		-	Laboratory	--	--	--	--	--	--
Volatile Organic Compounds										
Benzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Ethylbenzene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Styrene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Toluene	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Xylene, m+p-	E611A/WP	0.00040	mg/L	<0.00040	--	--	--	--	--	--
Xylene, o-	E611A/WP	0.00030	mg/L	<0.00030	--	--	--	--	--	--
Xylenes, total	E611A/WP	0.00050	mg/L	<0.00050	--	--	--	--	--	--
Hydrocarbons										
F1 (C6-C10)	E581.F1/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F2 (C10-C16)	E601/WP	0.10	mg/L	<0.10	--	--	--	--	--	--
F1-BTEX	EC580/WP	0.100	mg/L	<0.100	--	--	--	--	--	--
Bromobenzotrifluoride, 2- (F2-F4 surrogate)	E601/WP	1.0	%	70.3	--	--	--	--	--	--
Dichlorotoluene, 3,4-	E581.F1/WP	1.0	%	89.2	--	--	--	--	--	--
Bromofluorobenzene, 4-	E611A/WP	1.0	%	103	--	--	--	--	--	--
Difluorobenzene, 1,4-	E611A/WP	1.0	%	111	--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons										
Acenaphthene	E641A/WT	0.010	µg/L	<0.010	--	--	--	--	--	--

Analyte	*Method/Lab*	*LOR*	*Unit*	WP2501636-003 (Continued)		--	--	--	--	--	--
Polycyclic Aromatic Hydrocarbons - Continued											
Acenaphthylene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Acridine	E641A/WT	0.010	µg/L	<0.039	DLM RRR	--	--	--	--	--	--
Anthracene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benz(a)anthracene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benzo(a)pyrene	E641A/WT	0.0050	µg/L	<0.0050		--	--	--	--	--	--
Benzo(b+j)fluoranthene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benzo(b+j+k)fluoranthene	E641A/WT	0.015	µg/L	<0.015		--	--	--	--	--	--
Benzo(g,h,i)perylene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Benzo(k)fluoranthene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Chrysene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Dibenz(a,h)anthracene	E641A/WT	0.0050	µg/L	<0.0050		--	--	--	--	--	--
Fluoranthene	E641A/WT	0.010	µg/L	0.026		--	--	--	--	--	--
Fluorene	E641A/WT	0.010	µg/L	0.016		--	--	--	--	--	--
Indeno(1,2,3-c,d)pyrene	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
Methylnaphthalene, 1+2-	E641A/WT	0.015	µg/L	0.063		--	--	--	--	--	--
Methylnaphthalene, 1-	E641A/WT	0.010	µg/L	0.025		--	--	--	--	--	--
Methylnaphthalene, 2-	E641A/WT	0.010	µg/L	0.038		--	--	--	--	--	--
Naphthalene	E641A/WT	0.050	µg/L	<0.050		--	--	--	--	--	--
Phenanthrene	E641A/WT	0.020	µg/L	0.040		--	--	--	--	--	--
Pyrene	E641A/WT	0.010	µg/L	0.050		--	--	--	--	--	--
Quinoline	E641A/WT	0.050	µg/L	<0.050		--	--	--	--	--	--
B(a)P total potency equivalents [B(a)P TPE]	E641A/WT	0.010	µg/L	<0.010		--	--	--	--	--	--
PAHs, high molecular weight (BC AWQ)	E641A/WT	0.030	µg/L	0.076		--	--	--	--	--	--
PAHs, low molecular weight (BC AWQ)	E641A/WT	0.060	µg/L	<0.060		--	--	--	--	--	--
PAHs, total (CCME sewer 18)	E641A/WT	0.070	µg/L	0.195		--	--	--	--	--	--
PAHs, total (EPA 16)	E641A/WT	0.065	µg/L	0.132		--	--	--	--	--	--
Chrysene-d12	E641A/WT	0.1	%	102		--	--	--	--	--	--
Naphthalene-d8	E641A/WT	0.1	%	108		--	--	--	--	--	--
Phenanthrene-d10	E641A/WT	0.1	%	116		--	--	--	--	--	--

Please refer to the General Comments section for an explanation of any result qualifiers detected.

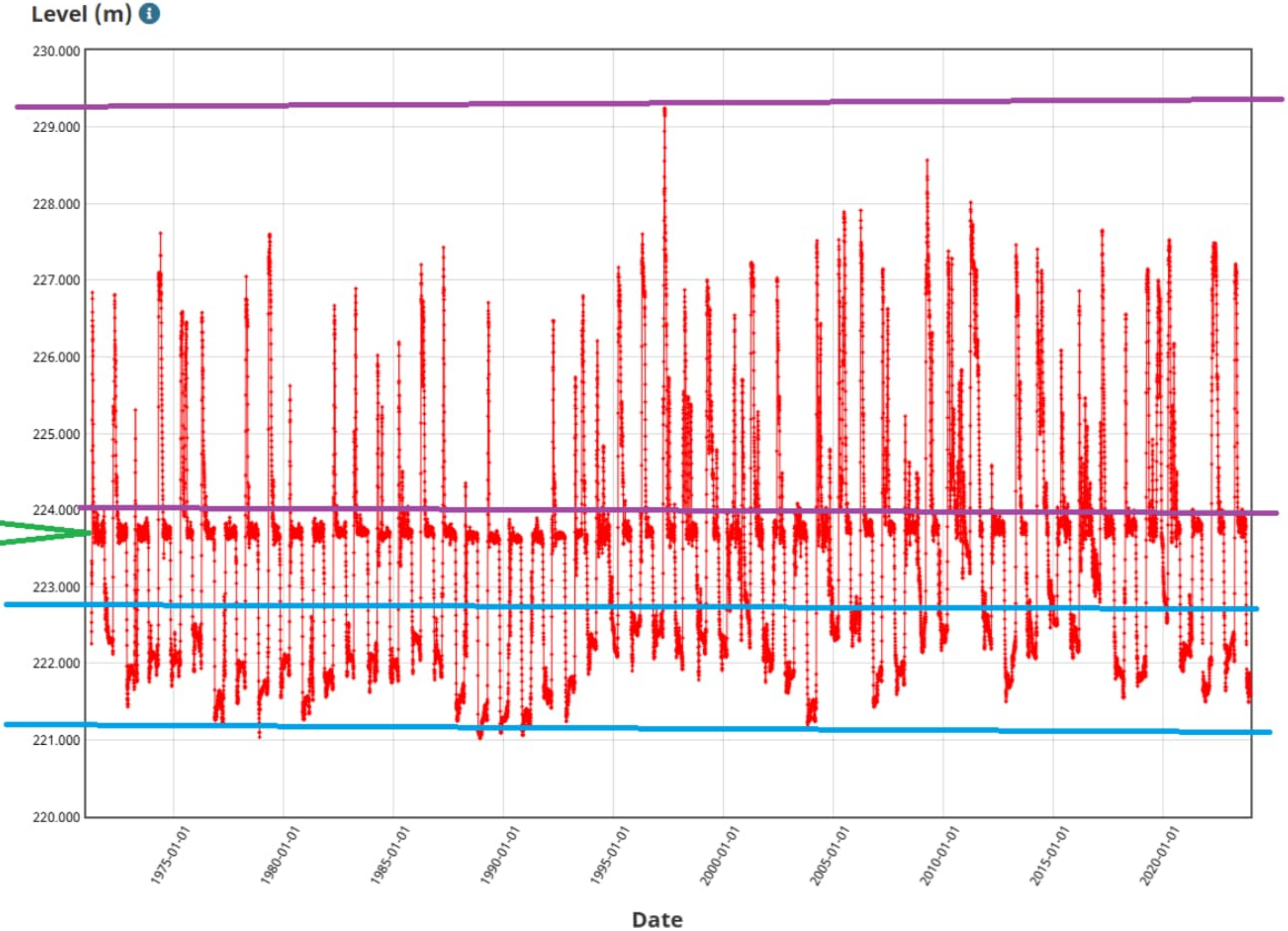
Please refer to the Accreditation section for an explanation of analyte accreditations.

No Breaches Found

Key:

Appendix III

Daily Water Level Grap



1929 DATUM

High variability in Spring

Spring

Summer levels very consistent. Controlled by St. Andrews Lock and Dam

Summer

Winter

Small variability in Winter

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