AECOM

NEWPCC Upgrade: Biosolids Facilities Early Works

CPR Crossing Geotechnical Report – FINAL – Rev. 2
900 mm LDS

WSTP

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June 2025

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AECOM: 2015-04-13

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1. Introduction

AECOM was retained by the City of Winnipeg to design and construct a new land drainage sewer (LDS) system. This system will connect the future Parcel B LDS System to the John Black outfall within the future Chief Peguis Trail right of way, near the North End Water Pollution Control Center (NEWPCC) site, as part of the NEWPCC Biosolids Facilities Project. The proposed LDS will cross under the Canadian Pacific (CP) railway at Mile 3.84 within the Winnipeg Beach Subdivision.

The proposed LDS qualifies as CP's "Process 2 – Intermediate" for the design, excavation, and construction criteria as outlined in their protocol: "CP Geotechnical Protocol for Pipeline and Utility Crossing Under Railway Tracks". According to CP's protocol, Process 2 is applicable to those crossing(s) applications that do not meet the conditions of Process 1, which depends on the pipe dimension, depth of pipe, and excavation and construction method. Further details on the general requirements of CP Protocol's Process 2 – Intermediate and proposed design parameters are provided in **Table 5-1** in Section 5.1.

The full Geotechnical Data Report (GDR) can be found in **Appendix I – NEWPCC Upgrade: Biosolids Facilities Early Works Geotechnical Data Report (Land Drainage System Crossing)** where it provides the findings of geotechnical investigation completed in NEWPCC, Winnipeg, Manitoba and characterizes the subsurface and groundwater conditions.

This geotechnical report has been prepared specifically for the proposed LDS crossing of the CP rail line located at Mile 3.84 within Winnipeg Beach Subdivision and provides comments and recommendations for consideration in the design and construction of the proposed LDS. German Leal will be the Geotechnical Engineer of Record (GER) for this proposed crossing.

Furthermore, the purpose of this report is to satisfy CP's Process 2 – Intermediate application requirements as outlined in the document titled "CP Geotechnical Protocol for Pipeline and Utility Crossing Under Railway Tracks," dated May 15, 2024 (Protocol). The settlement monitoring plan presented herein is to monitor the ground movement at the Canadian Pacific Railway (CPR) tracks before, during and after installation of the proposed LDS. This is to identify if the installation of the LDS results in the ground disturbance that could potentially affect the railway.

2. Proposed Site and Proposed Construction

The site at 2230 Main Street, Winnipeg, MB, is part of the North End Sewage Treatment Plant (NEWPCC). The terrain includes open fields, grass, and sparse trees. Testholes TH24-02 and TH24-08 are located along the future Chief Peguis Trail (CPT), which intersects the existing CP railway. These testholes were drilled on the west and east sides of the CP railway, in areas of tall grass.

The proposed LDS carrier consists of a 914 mm nominal diameter PVC pipe with an outside diameter of 972 mm. It is enclosed in a steel casing with a nominal diameter of 1184 mm and an outside diameter of 1219 mm.

The crossing profile indicates that at the railway crossing location, the elevation and depth of the following:

- The base of rail (BOR) track is at elevation Elev. 231.55 m.
- The invert elevation of the 1219 mm steel casing ranges from Elev. 226.279 m at the jacking pit to Elev. 226.236 m at the receiving pit. This results in an invert depth from the BOR of 5.28 m (Elev. 226.27 m).
- The depth from the BOR to the top of the steel casing is 4.06 m (Elev. 227.49 m).

The dimensions of the jacking and receiving pits are indicated in Drawing No. 1 in **Appendix II – Design Drawings**. The drawing indicates that a jacking pit and receiving pit will be constructed east and west of the crossing. These pits will be outside the CP railway right of way (ROW) and will be excavated within the future Chief Peguis Trail Right of way, north of NEWPCC site. In addition, the pit locations meet the CP requirements regarding the zone of influence (ZOPTL).

The bottom of the jacking pit and receiving pit is at Elev. 225.76 m with the depth of 5.79 m from the BOR to the bottom of the pits. Based on the drawing, the dimensions of the pits as follows:

- The length and width of the jacking pit are approximately 12.0 m and 4.95 m, respectively.
- The length and width of the receiving pit are approximately 4.95 m and 4.95 m, respectively.

3. Geotechnical Subsurface Investigation

As part of the CPR application, AECOM performed a geotechnical subsurface investigation in the future Chief Peguis Trail right of way near the NEWPCC to support the design and construction of the proposed 914 mm PVC LDS carrier.

The subsurface and drilling program was conducted on October 13, 2023, and December 15, 2023. Drilling services were provided by Paddock Drilling under the supervision of AECOM geotechnical field personnel. The field investigation consisted of using a 125 mm solid stem auger to drill two (2) testholes (identified as TH23-02 and TH23-08) to support the design and construction of the proposed LDS alignment. Testhole TH23-02 was drilled east of the CPR track at a depth of 10.7 m in fat clay, while testhole TH23-08 was drilled west of the CPR track at auger refusal at a depth of 17.1 m on very dense till. One (1) SP was installed in TH23-08 within the silt till layer at a depth of 17.0 m (Elev. 213.43 m).

During testhole drilling, subsurface conditions were visually observed and documented by AECOM geotechnical personnel using the Unified Soil Classification System (USCS). Soil samples were collected at intervals of 0.3 m to 1.5 m directly from the auger flights. The undrained shear strength of cohesive soils was assessed with a field torvane, and four undisturbed samples were retrieved using thin-walled Shelby tubes. Additionally, split spoon samples were taken from the underlying silt till material to obtain Standard Penetration Test (SPT) “N” values. Groundwater, seepage, and sloughing conditions were recorded upon completion of drilling.

A laboratory testing program was developed to measure the index properties of the different soil types encountered. This program included geotechnical testing on disturbed grab samples, split spoon samples, and undisturbed Shelby tube samples. The geotechnical laboratory testing was performed on selected soil samples to evaluate their physical characteristics, assess engineering properties, and aid in further characterization of the subsurface conditions. The testing program included determinations of moisture content, Atterberg Limits, grain size distribution using the hydrometer method, and unconfined compressive strength. Details regarding the laboratory testing program and the results are provided in the Geotechnical Data Report (GDR).

The Geotechnical Data Report is provided in **Appendix I**.

4. Subsurface Conditions

The soils encountered during the investigation consisted of topsoil at the ground surface underlain by fill (clay fill), fat clay, lean clay (with high silt content), then another layer of fat clay then silt till. Details regarding subsurface conditions are provided in Section 4 of the GDR, which is provided in **Appendix I**.

It should be noted that the silt content of lean clay based on the results of grain size analysis ranged between 79.8% to 84.5% indicating that the lean clay has high silt content.

4.1 Inferred Subsurface Stratigraphy

The ground surface for testhole TH24-02 is 230.43 m ASL and was terminated at a depth of 10.7 m in fat clay. **Table 4-1** summarizes depth, elevation and soil layers encountered during the investigation of testhole TH24-02.

Table 4-1: TH24-02 Inferred Subsurface Stratigraphy

Depth (m)	Elevation (m ASL)	Stratigraphy
0.0 – 0.5	230.43 – 229.93	Topsoil
0.5 – 1.5	229.93 – 228.93	Stiff fat clay
1.5 – 3.0	228.93 – 227.43	Soft lean clay
3.0 – 10.7	227.43 – 219.73	Firm fat clay

The ground surface for testhole TH24-08 is 230.29 m ASL and was terminated at a depth of 17.1 m in silt. **Table 4-2** summarizes depth, elevation and soil layers encountered during the investigation of testhole TH24-08.

Table 4-2: TH24-08 Inferred Subsurface Stratigraphy

Depth (m)	Elevation (m ASL)	Stratigraphy
0 – 0.3	230.29 – 229.99	Topsoil
0.3 – 1.1	229.99 – 229.19	Clay fill
1.1 – 2.0	229.19 – 228.29	Stiff fat clay
2.0 – 2.3	228.29 – 227.99	Soft lean clay
2.3 – 16.2	227.99 – 214.09	Firm fat clay
16.2 – 17.1	214.09 – 213.19	Silt till

4.2 Groundwater Conditions

Groundwater level was monitored later in January and October 2024 from the SP installed, as provided in Section 5.1 of the GDR. The depths of the groundwater level below ground surface were 1.38 m, 1.42 m, and 1.62 m at elevations Elev. 228.91 m, Elev. 228.870 m, and 228.680 m, respectively.

5. LDS Design Criteria

This report aims to support adequate subsurface information for crossing design, including geotechnical and groundwater information, to support the CP crossing application. CP's geotechnical or a CP approved services provider will assess the geotechnical information prior to receiving CP's approval for each crossing application.

5.1 Design Requirements

Based on the CP Protocol Requirements for pipeline crossing under railway tracks with an outside diameter of 300 mm to 1500 mm. The proposed 972 mm OD PVC LDS Carrier is categorized as "Process 2 – Intermediate". **Table 5-1** provides the CPR Protocol Requirement for Process 2 – Intermediate and the proposed trenchless crossing based on the project drawing in **Appendix II**.

Table 5-1: CPR Protocol Requirement and Proposed Design Parameter

Criteria	CPR Protocol Requirement (1)	Proposed Crossing Design
Dimension Criteria		
Outside Pipe Diameter	300 mm to 1500 mm	**PVC**: 972 mm outside diameter **Steel Casing:** 1219 mm outside diameter
Cover between BOR and top of pipe	Greater than 1.5 m or 2 pipe diameter whichever is greater	4.06 m (Elev. 227.49 m)
Adjacent structures including switches	Within 2.5 times, cover between BOR and top of pipe	None
Depth of pipes outside Zone of Potential Track Loading (ZPTL)	Less than 0.91 m burial within ZPTL	Approximate depth of pipe within ZPTL (west) = 2.46 m Approximate depth of pipe within ZPTL (east) = 2.49 m
Excavation Criteria		
Excavation close to CP track(s)	Excavation or jacking/access pits within 10 m of the closest track centreline	Centre line of track to edge of Jacking Pit: 20.18 m Centre line of track to edge of Receiving Pit: 20.15 m Outside ZPTL and CP's ROW
Crossing Angle	More than 45 degrees off perpendicular to the track	71°
Construction Method Criteria		
	Auger Boring and Pipe Ramming	Guided Auger Boring with pipe jacking or Guided Pipe Ramming
Other Criteria		
Settlement for Class 2 track	Level 1 Alert - (Review Threshold): >11 mm Level 2 Critical - (Stop Work): >22 mm	Provided in Section 7.2
Approximate Length of Crossing	None	42.3 m

(1) CP Geotechnical Protocol for Pipeline and Utility Crossing(s) under Railway Tracks Criteria for Process 2 – Intermediate.

6. Carrier Pipe Installation

6.1 Anticipated Stratigraphy

The proposed 1219 mm OD steel casing invert ranges in elevation from Elev. 226.236m at the east pit to Elev. 226.279 m at the west pit. The soils encountered in testholes are as follows:

- TH23-02 (east of rail line) consisted of topsoil at the ground surface underlain by fat clay, lean clay, and another layer of fat clay.
  Particularly, the second layer of fat clay was encountered at Elev. 227.380 m and extended down to the termination depth of 10.7 m (Elev. 219.730 m).
- TH23-08 (west of rail line) consisted of topsoil at the ground surface underlain by fill (clay fill), fat clay, lean clay, and another layer of fat clay then silt till.
  Particularly, the second layer of fat clay was encountered at Elev. 228.290 m and extended to Elev. 214.140 m.

Table 6-1 provides the anticipated soil stratigraphy along the proposed LDS trenchless bore path.

Table 6-1: Anticipated Stratigraphy along the LDS Trenchless Bore Path

<table>
<tr><th>Proposed LDS Bore Path</th><th>Approximate Elevation of LDS at BOR (m)</th><th>Anticipated Soil Unit at the Jacking Pit</th><th>Anticipated Soil Unit at the Receiving Pit</th><th>Elevation of Soil Unit (m)</th></tr>
<tr><td>Top of Carrier</td><td>227.49</td><td>Fat Clay (CH)</td><td>Fat Clay (CH)</td><td rowspan="2">TH23-02 Fat Clay below Elev. 227.38 m

TH23-08 Fat Clay between Elev. 228.0 m to Elev. 214.14 m</td></tr>
<tr><td>Bottom of Carrier</td><td>228.71</td><td>Fat Clay (CH)</td><td>Fat Clay (CH)</td></tr>
</table>

As shown in **Table 6-1**, it is anticipated that the proposed steel casing will be within the fat clay layer at the jacking pit and the receiving pit.

The depth from the BOR to the bottom of the jacking pit and receiving pit is 5.79 m at elevation Elev. 225.76 m. Based on the bottom depth of the pits, it is anticipated that the topsoil, fill, lean clay and fat clay layers will be encountered during excavation of the jacking and receiving pit.

6.2 Tunnelman's Ground Classification

Table 6-2 is provided for completeness and as general information for the anticipated ground conditions along the crossing alignment. This table provides the framework for Tunnelman's Ground Classification and indicates the respective tunnel working conditions for reference as outlined by Heuer and Virgins (1987) and Brandt (1970) and others. Soft to firm fat clay below the groundwater level is anticipated to exhibit a 'squeezing' behavior.

Table 6-2: Tunnelman's Ground Classification and Probable Work Conditions

Classification		Representative Soil Types	Tunnel Work Conditions
Hard		Very hard calcareous clay; cemented sand and gravel.	Tunnel heading may be advanced without roof support.
Firm		Loess above water table; hard clay, marl, cement sand and gravel when not highly overstressed.	Tunnel heading can be advanced without initial support, and final lining can be constructed before ground starts to move.
Raveling	Slow Raveling	Residual soils or sand with small amounts of binder may be fast raveling below the water table, slow raveling above. Stiff fissured clays may be slow or fast raveling depending upon degree of overstress.	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed, due to loosening or to overstress and “brittle” fracture (ground separates or breaks along distinct surfaces, opposed to squeezing ground). In fast raveling ground, the process starts within a few minutes, otherwise the ground is slow raveling.
	Fast 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 anhydrite.	Like squeezing ground, moves slowly into tunnel, but movement is associated with a very considerable volume increase in the ground surrounding tunnel.
Running	Cohesive Running	Cohesive running occurs in clean, fine moist sand.	The removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a “run” whereby the material flows like granulated sugar until the slope angle becomes equal to about 34°. If the “run” is preceded by a brief period of raveling, the ground is called cohesive raveling.
	Running	Running occurs in clean, coarse or medium sand above the GWT.	
Very Soft Squeezing		Clay and silts with high plasticity index.	Ground advances rapidly into the tunnel is plastic flow.
Flowing		Below the water table in silt, sand, or gravel without enough clay content to give significant cohesion and plasticity. May also occur in highly sensitive clay when such material is disturbed.	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 filled.
Bouldery		Boulder glacial till; rip-rap fill; some landslide deposits; some residual soils. The matrix between boulders may be gravel, sand, clay or combination thereof.	Problems occurred in advancing shield or in forepoling; blasting or hard mining ahead of machine possibly necessary.

Since we are expecting fat clay, it is anticipated that the Tunnelman’s ground classification will range from firm to slow raveling.

6.3 Steel Casing

The steel casing size for the LDS is 1219 mm in outside diameter (OD), as proposed by others. For a comprehensive settlement estimate, a 25.4 mm overcuts is considered.

The following table summarizes the details regarding the casing size, its description, and the corresponding elevations at the east and west pits invert elevations:

Table 6-3: Invert Elevations for Various Casing Sizes

Casing Size (OD)	Description	Invert Elevations (East Pit)	Invert Elevations (West Pit)
1219 mm	Steel case	226.236 m	226.279 m

It is the contractor's responsibility to determine the appropriate method for steel casing installation. CPKC will be informed of the method of installation if it differs as its described in this report.

6.4 Recommended Installation Options

The methods of steel casing installation that are considered are:

- Guided Auger boring with pipe jacking (with soil plug) (also called Pilot tube Guided Auger bore).
- Guided Pipe ramming (also called Pilot tube guided Pipe ramming).

Both methods offer similar face support at the lead end of the steel casing. The steel casing would have an OD of 1219.2 mm to accommodate the LDS PVC 972 mm outside diameter (OD). The casing should be installed with a guided pilot tube when auger boring or pipe ramming method is used. The pilot tube method should utilize a guided pilot tube as a technique for accurately installing a pipe to line and grade. The pilot tube installation serves as the initial step in guided boring technology. Although it is recommended, it is the responsibility of the Contractor to determine the need for guided pilot tubes.

These methods have been considered based upon the known availability of resources, equipment, and expertise within the Manitoba market. Other factors for consideration including the geotechnical/geological constraints are discussed in Section 6.5.6 of this report. The contractor shall submit a trenchless installation work plan for the Geotechnical EOR to review. A copy of the reviewed trenchless installation work plan will then be provided to CPKC.

6.4.1 Guided Auger Boring with Pipe Jacking Method

The guided auger boring with pipe jacking method involves several key steps. First, excavate the trench or pit to create the launching/jacking area. Next, pilot tubes are installed to control line and grade. The steel casing is connected to the installed pilot tubes and jacked into place while a soil plug is maintained to provide face stability. The auger is used inside the casing to bore through the soil, with soil cuttings removed towards the launching/jacking pit. Throughout this process, the soil plug is maintained for continuous face stability and reduce the potential for ground subsidence. This method provides accurate control of line and grade and helps identify and mitigate potential obstructions before advancing the casing.

6.4.2 Guided Pipe Ramming

Pipe ramming is a trenchless construction method whereby a pneumatic hammer is used to drive the steel casing through the ground, and spoils are removed from the inside of the casing. With the guidance of pilot tube (Guided pipe ramming), this method combines the line and grade accuracy of the pilot tube installation with the power of compressed air pneumatic pipe rammer affixed to the rear of the casing. Benefits of this method are that the casing can be advanced through poor soils with minimal surface effects, by maintaining a soil plug within the pipe to control caving or flowing soils. Overcut is minimal (typically less than 25 mm) and can be used to reduce friction of the drive when used in conjunction with lubricants and a correctly specified hammer.

6.5 Trenchless Construction Risks

Each trenchless option for the CP rail crossing has been evaluated against the following risks:

Table 6-4: Evaluation of Trenchless Construction Risk

Trenchless Method	Perceived Risk
Guided Auger boring with pipe jacking	Ground settlement and heave
	Buried Obstructions
	Groundwater
	Pipe alignment/grade control
	Dense/very stiff soil conditions
Guided Pipe Ramming	Ground settlement and heave
	Buried Obstructions
	Groundwater
	Pipe alignment/grade control
	Noise and vibrations

6.5.1 Ground Settlement and Heave

The major advantage of guided auger boring and guided pipe ramming methods is the reduced ground disturbance during installation. However, ground settlement and heave can still occur during installation of the steel casing.

Minor groundwater seepage was observed in testhole TH23-02 at a depth of 1.5 m (Elev. 228.93 m), and moderate groundwater seepage was observed in testhole TH23-08 at a depth of 2.0 m (Elev. 228.29 m). The source of seepage was observed from the lean clay with high silt content. Soil sloughing was observed in testhole TH23-08 in soft fat clay at a depth of 13.7 m (Elev. 216.59 m), while no soil sloughing was observed in testhole TH23-02 during or upon completion.

It should be noted that soil sloughing may be encountered in soils with:

- Moisture content closer to its liquid limit – indicating that the soil is behaving like liquid.
- High silt content – silts are fine soils.
- Undrained shear strength less than 25 kPa i.e., soft soils.

Based on the results of the laboratory test results and the geotechnical investigation:

- Moisture content of the lean clay ranged between 23.6% to 25.2% with its liquid limit ranging between 23% to 25%, indicating that lean clay’s moisture contents are close to their liquid limit.
- Silt content of the lean clay ranged between 79.8% to 84.5%, indicating that the lean clay had high silt content.

Although soil sloughing was not observed from the lean clay layer during or upon drilling, soil sloughing may be encountered during construction as noted above.

Surface heave can occur during installation using pipe jacking by auger boring if the casing is advanced through the fat and lean clay soil too quickly without allowing time for the auger to remove the displaced soils. Settlement can also occur if flowing soils enter the casing. For auger boring, the management and control of support pressures at the leading face of the tunnel is largely dependent upon the plug of soil formed in front of the auger and casing.

Heave can also occur if the soil within the encasement pipe forms a soil plug during installation causing the soil in front of the pipe to displace as the encasement pipe continues to move forward. The potential for soil plugging varies depending on the installation technique and is more likely to occur for methods that involve jacking or ramming. AECOM recommends that a soil plug is developed at the leading face of the trenchless tunnel, however, advancement of the casing should be conducted at a pace so that soil can advance into the casing and be removed as necessary. If the casing is advanced too quickly, the material in front of the soil plug can build up, resulting in heave at the ground surface.

6.5.2 Buried Obstructions

Buried obstructions were not encountered during AECOM's geotechnical investigation in October 2023 and December 2023. However, buried obstructions such as abandoned rail ties and abandoned pipe may be encountered during trenchless method. Encountering buried obstructions can prevent or slow down the progress of a trenchless method. Particularly, auger boring with pipe jacking method can have difficulty cutting and moving obstructions beyond the auger flights, potentially creating misalignment to the pipe jacking An installation technique should be selected that can accommodate removal of potential obstructions without having to remove or expose the leading edge of the carrier pipe. Prior to construction, any utilities and/or fiber optics within the ROW should be located by the contractor.

6.5.3 Groundwater

Groundwater seepage and sloughing conditions were recorded upon completions of drilling of each testhole. As mentioned in Section 5 of the GDR, minor groundwater seepage was observed in testhole TH23-02 at a depth of 1.5 m (Elev. 228.93 m), and moderate groundwater seepage was observed in testhole TH23-08 at a depth of 2.0 m (Elev. 228.29 m). The source of seepage was observed from the lean clay with high silt content. Final groundwater level was observed at a depth of 7.5 m (Elev. 222.93 m) upon completion of drilling in testhole TH23-02. Since groundwater seepage was observed during drilling, the contactor should be prepared to deal with groundwater infiltration originating from the lean clay.

A standpipe piezometer (SP) was installed in testhole TH23-08 slotted within the silt till layer to monitor and measure the groundwater level in the testhole. Groundwater level was monitored later in January 24, January 26 and October 24, 2024, from the SP installed, as provided in Section 5.1 of **Appendix I**. The depths of the groundwater level below ground surface were 1.38 m, 1.42 m, and 1.62 m at elevations Elev. 228.91 m, Elev. 228.87 m, Elev. 228.68 m. It should be noted that the groundwater elevations from the SP installed in testhole TH23-08 are higher than the groundwater elevation encountered within the study limits and may not be representative of the groundwater elevation. The installation of the PVC LDS carrier (top of carrier Elev. 227.49 m) is below the groundwater elevation recorded by the standpipe piezometer (SP) installed within the study limits. The groundwater recorded is equivalent to approximately 1.85 m of total head within the jacking and receiving pit. The contractor should be prepared to deal with groundwater during installation and during excavation of the jacking and receiving pit. The contractor should have adequate pumping to maintain safe excavation.

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.

Groundwater will require careful management and control throughout the carrier installation process regardless of which trenchless method is adopted. Groundwater can promote instability at the face of the carrier 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 pipe jacking, pipe jacking, or other techniques are elected by the contractor. The contractor shall submit a dewatering plan for the Geotechnical EOR to review. A copy of the reviewed dewatering plan will then be provided to CPKC. The contractor will be provided with commentary regarding risk associated with creating settlement on the track due to dewatering.

6.5.4 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 rail ties or abandoned pipes), encountering an obstruction may result in the reduction of alignment and grade control accuracy. The steel casing should be installed using a guided pilot tube when auger boring or pipe ramming methods are used. The use of guided pilot tube provides an accuracy of +/- 25 mm (1 inch) from the design grade and +/- 76 mm (3 inches) from the design alignment at any location.

In the case of pipe ramming without using guided pilot tube, alignment and grade control is not readily steerable and can be significantly affected by ground conditions. Typical accuracies are in the order of 1 % of the drive length, although with good initial alignment and control of the lead section, accuracies can be increased from 0.1% to 0.5%.

6.5.5 Void Development

As mentioned in Section 6.1, the proposed LDS (i.e., PVC LDS carrier pipe) is anticipated to be within the fat clay layer encountered in testholes TH23-02 and TH23-08. Voids between the carrier pipe and the bore may develop as the auger advances along its drive length. Void development is more prevalent in cohesive soils (i.e., firm to very stiff clays) which generally may provide the ability to support an open excavation without collapsing immediately under pressure of the above soil. Over time, this void may reduce due to settlement, swelling or softening of the exposed soils leading to collapse. Circumference grouting outside the carrier maybe required if these ground conditions are encountered. The contractor should install the entry and exit seal on the break-in and break-out point of the trenchless crossing, respectively, to prevent slurry loss prior to grouting.

6.5.6 Horizontal Stresses due to Pipe Jacking on the Casing Pipe

In general, the jacking force required to propel the pipe sections forward must overcome forces associated with face pressures on the cutting head, plus friction on the shield and pipeline. The frictional forces develop between the surrounding soil and the exposed outer surface of the shield and installed pipe sections. The face pressure component relates to the depth of burial and can be estimated based on the soil and groundwater conditions at the site. The face pressure component of the jacking force remains theoretically constant if the depth of soil cover over the pipeline is constant. However, the frictional force increases as the drive length increases. As a result, longer drives require greater jacking forces. Other construction issues such as pipe misalignment due to obstructions and jacking stoppage can also affect the required jacking force.

6.5.7 Pipe Ramming Dynamics

To drive the casing pipe horizontally along the proposed alignment, the pipe ramming force must overcome soil resistance forces (as discussed in Section 6.5.6). Wave equation analysis should be performed to optimize the hammer energy required to install the pipe without damage. The maximum energy transfer from the hammer to the pipe is dependent upon hammer selection, hammer alignment, and the degree of tension on restraining chains. Total soil resistance generally increases with pipe length, depth of soil cover and increasing soil strength.

Wave Equation Analysis can be performed upon selection of an appropriate hammer type prior to construction. It is the responsibility of the Contractor to conduct the Wave Equation Analysis, so that an appropriate hammer type can be selected.

6.5.8 Face Stability

Based on the results of the 2023 AECOM geotechnical investigation and the proposed LDS profile, the proposed LDS carrier will be installed within a fat clay layer. Excavation in front of the leading carrier length will cause stress relief unless support is provided to retain the exposed face. As discussed in Section 6.5.1 of this report, mitigation measures should be put in place to limit the loss of ground at the face of the PVC LDS carrier.

It is anticipated that installation of the carrier will take place below the groundwater table; therefore, reduced face stability is considered likely along the LDS drive length.

Two extreme cases of failure may occur due to the poor management of face support pressure: the formation of chimneys or the development of blow-outs in the ground above the tunneling face. The minimum pressure to avoid face instability is affected by various factors, such as cohesion, friction angle and permeability of the ground, type of the machine, advance rate, unit weight of slurry or conditioned soil, tunnel diameter, cover depth, and depth of the groundwater table.

7. Geotechnical Assessment

7.1 Jacking Pit and Receiving Pit

According to CPR Geotechnical Protocol for Pipeline and Utility Installations within Railway Right of Way, the location of the jacking or receiving pits shall not extend into the "Zone of Potential Track Loading" (ZPTL). The zone of loading is considered the area under the track and within a 1.0(V):1.5(H) soil zone extending down from a point at the level of the BOR and 2.0 m from the centreline of the tracks. The face of the east pit is 20.15 m from the centreline of the track and the face of the west pit is 20.18 m from centreline of track. In this case, the excavations required to construct the pits (i.e., jacking and receiving pits) will not extend into the ZPTL and will be located outside CP's ROW. Further details about the locations of the jacking and receiving pit are indicated on Drawing No.1 in **Appendix II**.

According to the CPR Geotechnical requirement criteria for excavation close to CP track(s), when the excavation of jacking/receiving pits are within 10 m of the closest track centreline, the excavation criteria fall under Process 2 – Intermediate. In our case, the excavation of the jacking and receiving pit will not be within 10 m of the closest track centreline.

As mentioned in the previous sections, the depth from the BOR to the bottom of the jacking pit and receiving pit is 5.79 m at elevation Elev. 225.76 m. The soils encountered in testholes TH23-02 and TH23-08 consisted of topsoil at the ground surface underlain by fill (clay fill), fat clay, lean clay, another layer of fat clay then silt till. It is anticipated that during the excavation of the pits that the topsoil, fill, lean and fat clay will be encountered.

As mentioned in previous sections, sloughing may occur during the excavation of pits in soils with moisture content close to their liquid limit (i.e., behaving like a liquid), high silt content, and soft soils such as lean clay and soft fat clay. Based on the results of the geotechnical investigation, the undrained shear strength of the fat clay ranged from 9.8 kPa to 68.7 kPa, generally decreasing with depth, classifying the material as stiff to soft in consistency. As the excavation of the pits gets deeper into the fat clay, the material becomes progressively softer, which may cause sloughing.

Due to potential variations in in situ soil conditions between test holes, caution should be exercised during construction. It is advisable to consider the use of large excavating equipment to achieve the intended excavation depth safely and efficiently.

Based on the depth of the jacking and receiving pit, it is anticipated that temporary shoring will be used to facilitate excavation of the jacking and receiving pits. The pits need to be appropriately shored (in accordance with applicable regulations) since the side walls are normally cut vertically into the soil to conserve space. The pits should be large enough to accommodate the backstop, jacking equipment, spacer, muck removal equipment, lubricant pumps, lines, pneumatic hammers, and augers, etc. Additionally, the pits should also have walking room on each side of the jacking/pneumatic equipment. All equipment is normally centred along the centre line of the carrier pipe.

7.1.1 Excavation

Pipe jacking operations require the excavation of a suitable jacking and receiving pit. AECOM should be contacted to observe the materials excavated from the jacking and receiving pits and confirm soil conditions match to those encountered during the field drilling program. The method of excavation and support of excavation sidewalls are the responsibility of the contractor and must comply with the appropriate regulations under the Manitoba Workplace Safety and Health Act. The information provided below is for use by the owner and engineer and should not be interpreted to mean that AECOM is assuming responsibility for the contractor's actions or site safety.

Contractors should acknowledge these concerns and develop a Safe Excavation Plan accordingly. Side slopes for temporary open-cut excavation must conform to the Manitoba Guide for Excavation Work. According to Manitoba's Guide for Excavation Work, the minimum excavation side slope is 1H:1V from the base of the excavation. Services of a professional engineer is required to design support structures where the worker is required to enter any open excavation that exceeds 1.5 m in depth.

As mentioned from previous sections, groundwater seepage was observed in the lean clay layer during AECOM's drilling program. During excavation (i.e., short term duration), groundwater seepage may occur from the lean clay layer. The seepage from the lean clay is due to seasonal fluctuation. When the excavation is left open for a long period of time (i.e., long term duration), groundwater from the silt till layer may rise up. Thus, the contractor should be prepared to deal with groundwater during the excavation of the jacking and receiving pit. The stability of the excavation slopes should be monitored regularly by knowledgeable geotechnical personnel. Shoring related to temporary work is the responsibility of the contractor, and all necessary measures should be undertaken to protect against adverse detrimental impacts.

The contractor is responsible for the design and implementation of any required dewatering systems to facilitate safe and stable excavations. The design of the dewatering system would need to address the extent of dewatering required, the depth of the intended excavations, and the soil and groundwater conditions that prevail at the time of excavation. The program should contain a communication protocol with the regulatory agencies and the public, short-term containment, sampling and analysis, permitting, disposal, and reporting requirements. Typically, in the soil conditions present at the site, groundwater dewatering is very rarely required, and control of nuisance seepage and surface water is usually all that is required.

The contractor is solely responsible for the design and implementation of any required dewatering, including requirements for withdrawal, handling, treatment, and discharge in accordance with the Province of Manitoba requirements. For this project, it is anticipated that water seepage into the excavations could be sufficiently controlled using a perimeter ditch and sump. The contractor should submit the design of any required dewatering system to AECOM for the Geotechnical EOR to review. The contractor should submit any required documentations for the dewatering permit. A copy of the reviewed dewatering plan will then be provided to CPKC. The contractor will be provided with commentary regarding risk associated with creating settlement on the track due to dewatering.

7.1.2 Temporary Shoring

As mentioned in Section 7.1, it is anticipated that temporary shoring will be used to facilitate excavation of the jacking and receiving pits. Comments regarding the design and temporary shoring system are therefore provided as follows.

The design of the temporary shoring system should be carried out by a professional engineer specialized in shoring design. The shoring system should also be designed in accordance with the methods described in the Canadian Foundation Engineering Manual.

In consideration of the information provided in the preceding sections, it is anticipated that the maximum depth of the jacking and receiving pits will be in the order of 5.79 m (Elev. 225.76 m ASL) for the 972 mm OD PVC LDS carrier below the BOR. In consideration of the conditions encountered in the testholes, it is recommended that the design of a shoring system consider the parameters provided in **Table 7-1**. **Table 7-1** provides the recommended earth pressure coefficients, effective cohesion, angle of internal friction and bulk unit weight of the fill, silt, lean clay (CL), and fat clay (CH) for use in the calculation of lateral earth pressures. The bulk unit weight of the fat clay (CH) was taken from the average bulk unit weight obtained from AECOM's lab test results. The effective cohesion and friction angles provided in the table have been assumed based on the soil conditions encountered in the testholes and consideration of literature references for similar soils.

Table 7-1: Lateral Earth Pressure Design Parameters

USCS Soil Type	Soil Unit Weight (kN/m^3)	Effective Cohesion (c')	Angle of Internal Friction (°)	At Rest Lateral Earth Pressure Coefficient (K_o)	Active Lateral Earth Pressure Coefficient (K_a)	Passive Lateral Earth Pressure Coefficient (K_p)
Fill (Fat Clay)	17	5	25	0.58	0.41	2.46
Silt	16	0	28	0.53	0.36	2.77
Lean Clay	18	5	25	0.58	0.41	2.46
Fat Clay	17	3	25	0.58	0.41	2.46

For the purposes of design of shoring systems, it is recommended that the groundwater elevation be taken as 228.91 m ASL as being the highest elevation of the groundwater level recorded in the SP installed in TH24-08.

Given that the water table is observed at 228.91 m ASL. It should be noted that groundwater levels observed may not be representative of stable groundwater conditions. Seasonal fluctuations due to precipitation, snow melting, drainage conditions on site and other factors may influence the groundwater levels recorded over time. Therefore, groundwater conditions at the time of construction may vary from the recorded groundwater depths above. Construction dewatering should be expected to isolate the work zone and facilitate construction in dry conditions; therefore, provisions for dewatering and groundwater control should be accounted for in the project schedule and cost.

A perimeter ditch and associated pumping and an appropriate dewatering system should be provided to intercept surface runoff and groundwater from entering the excavation. To avoid the possibility of piping within the excavation, dewatering should be performed. The Contractor should submit a safe excavation plan, including dewatering measures, for Engineer review.

Monitoring must be carried out during installation/construction process and following installation/construction to confirm that movements of the temporary shoring system are within a pre-determined acceptable range.

7.1.3 Excavation Base Stability

According to the Canadian Foundation Engineering Manual (CFEM 5e), deep excavations in soft-to-firm clays are susceptible to base heave or squeezing failures due to soil overstressing in shear. For this project, braced excavations are planned for a pipe installation at the jacking and receiving pits at a depth of approximately 5.79 m BGS (225.76 m ASL) for the 972 mm OD PVC LDS carrier. If the soil below the excavation base is soft and normally consolidated, heaving may occur. Although the soil below the excavation base is firm to stiff and normally consolidated, heaving could still be a concern if the pits reach a soft clay layer. The soil above the base acts as a surcharge on the soil below, potentially exceeding its bearing capacity and causing heaving.

The Factor of Safety (FS) against base heave associated with soil squeezing or shear failure is calculated using the following equation:

$$(FS)_b = \left(\frac{N_b s_u}{\sigma_z + q}\right) = \left(\frac{N_b s_u}{\gamma H + q}\right) \qquad \textbf{Equation 1}$$

Where:

- $(FS)_b$ = factor of safety against base heave associated with soil squeezing or shear failure.
- N_b = stability factor dependent upon geometry of the excavation from Fig. 20.21 of CFEM. N_b depends on H/B and L/B (H is the bottom of the excavation, B is the width of the excavation and L is the length of the excavation).

- s_u = average undrained shear strength of soil below the base, corrected for plasticity, test method, and anisotropy as appropriate (kPa).
- σ_z = total overburden pressure at the bottom of the excavation:
  - γ = unit weight of fat clay. In this calculation, 17 kN/m^3 was used.
  - H = bottom of the excavation.
- q = surcharge on the surface. It has been assumed that no surcharge will be on the surface. Thus, q = 0 kPa.

For $(FS)_b$ less than 2, substantial deformations of the excavation support, base and surrounding ground may occur. If $(FS)_b$ is less than 1.5, the sheeting should be extended below the base of the excavation for stability. Wall movements, strut loads, and wall moments are sensitive to $(FS)_b$.

As previously mentioned, the base of the launch and retrieval shaft is with a firm to stiff fat clay (CH) layer. Base heave is deemed satisfactory if the factor of safety $(FS)_b$ is greater than 1.50. The calculated $(FS)_b$ for the excavation of the proposed jacking pit shaft is 1.73. The calculated $(FS)_b$ for the excavation of the proposed retrieval shaft is 2.49. Although heave of the excavated shafts provides satisfactory FS, the design of the temporary shoring system should be carried out by a professional engineer specialized in shoring design.

7.1.4 Buoyancy Uplift from Excess Groundwater Pressure Beneath an Impermeable Stratum

According to the Canadian Foundation Engineering Manual (CFEM 5e), when an excavation is dug into a clay deposit underlain by a pervious stratum under artesian pressure, pressure and seepage may result, leading to instability of the excavation. An analysis has been prepared for the design of the temporary excavation, excavation depth and piezometric condition within the underlying fat clay.

The basal heave analysis is based on the ratio of total stresses and uplift pore water pressure.

For this approach, the FS is expressed using the equation:

$$FS = \frac{H_C \gamma_C}{H_w \gamma_w}$$ **Equation 2**

Where:

- γ_c = unit weight of fat clay = 17 kN/m^3.
- H_c = thickness of the fat clay between the bottom of the excavation to the top of the glacial till = 11.60 m.
- γ_w = unit weight of water = 9.81 kN/m^3.
- H_w = the total head in the glacial till layer = 14.81 m.

According to the CFEM, heave due to artesian pressure is satisfactory if the Factor of Safety (FS) is greater than 1.1. The FS for the jacking shaft is 1.35. The testhole in the receiving shaft was not drilled to refusal on till, so it is assumed that conditions are similar to the jacking shaft, with an FS greater than 1.1. If the FS is less than 1.1, the contractor should develop a dewatering plan to manage artesian pressures. A professional engineer specializing in braced excavation design should be consulted before construction begins.

7.2 Settlement Estimation

Like other tunnelling methods, pipe ramming or auger bore will result in a change in the state of stress in the ground with corresponding settlements. Ground subsidence can be caused by several factors such as ground loss at the tunnel face, behind the tail of the shield and through the tunnel support or linings. Based on cohesive soils tending toward a stable tunnelling face, the only significant contribution to ground loss is the closure of the over-cut. The radial overcut (25.4 mm) is the annular space between the tunnel boring walls and the installed pipe. Some degree of ground subsidence can be expected from tunneling although in many instances its effects, from a practical perspective, are negligible with proper technique.

7.2.1 Empirical Method

A method for prediction of settlement that may develop due to trenchless installation is the method outlined by Schmidt (1969) and later by Thomson (1993). A ground surface deformation induced by trenchless construction is estimated using a reverse gaussian curve based on the anticipated ground loss.

The empirical method is characterized as a simplified method and an upper bound solution as the method does not consider the potential for arching effects in the overlying soil mass above the borehole obvert, nor does the method consider soil layering, groundwater conditions or the shape of the void.

This method assumes that the total ground loss (V_t) (or over-drill) that occurs over the pipe leads to settlement at the ground surface in the shape of a reverse gaussian curve (normal probability distribution). The maximum settlement δ_{max} occurs at the ground surface above the tunnel centreline and is estimated from the following equation:

$$\delta_{max} = \frac{V_t}{2.5i} \qquad \textbf{Equation 3}$$

Where "i" is the point inflexion for the normal distribution, and "V_t" represents the volume of ground loss during tunnel excavation multiplied by the cross-sectional area of the drilled shaft. The method suggests the following correlation between "i", depth of tunnel centreline (Z) and settlement trough parameter (K, function of soil type) for cohesive soil.

$$i = Kz \qquad \textbf{Equation 4}$$

Based on the conditions encountered in the testholes soil stratigraphy, the proposed auger boring path is anticipated to start in soft to stiff fat clay. The top portion of the auger boring path then transitions to lean clay with high silt content. The track subgrade is likely comprised of ballast material. However, the empirical method does not address multi-layer system. The method suggests K values ranging from 0.4 to 0.7 for very soft to stiff clay soils, 0.5 for normally consolidated soils, and a K value of 0.25 for cohesionless soils. The smaller the K value is taken the larger will be the settlement estimate. It is anticipated that the proposed LDS will start within the fat clay layer and transition to lean clay with high silt content. Given the conditions in the testholes within the fat clay layer and lean clay layer, a K value of 0.40 is considered for this estimation.

It is typical to assume contribution of 10% to 25% of the annular space to the ground surface deformation given the potential benefit from ground arching effects and localized ground loosening (i.e., volume change). In addition to the annular space, we can consider a ground loss of approximately 1% to 2.5% of the of the bore hole volume to occur at bore face for boring in soft to stiff cohesive soils (i.e., fat clay). In this respect, a combination of over-drilling (V_1) and soil raveling at the bore face (V_2) is considered to contribute to ground loss (V_t).

Figure 7-1 shows the settlement estimation for a permanent steel casing with an 1219 mm OD. The graph below present the results of settlement analysis based on the following:

- Scenario 1: 10% annular collapse with 1% soil loss.
- Scenario 2: 25% annular collapse with 2.5% soil loss.

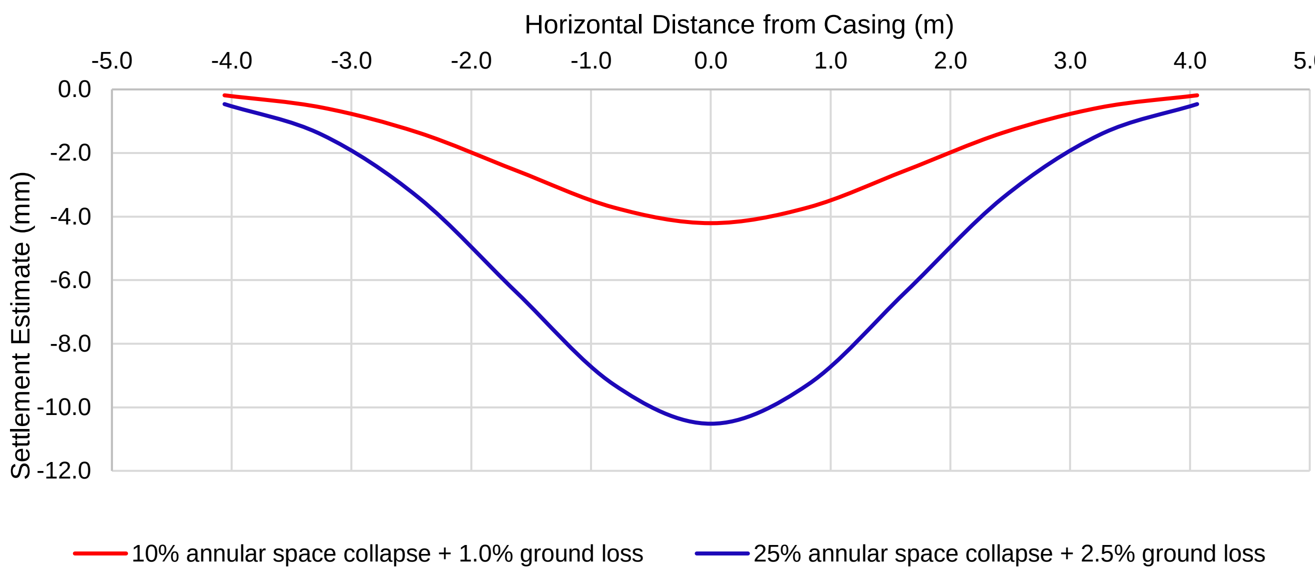

Figure 7-1: 10% Annular Collapse + 1.0% Ground Loss and 25% Annular Collapse + 2.5% Ground Loss

As shown in the **Figure 7-1,** the estimated settlement for a 1219 mm casing OD and a 10% annular space collapse with 1.0% ground loss is 4.2 mm, which is below the Level 1 “Alert - (Review Threshold, 11 mm)” and Level 2 “Critical - (Stop Work, 22 mm)” settlement limits for the CP Class 2 track. While the estimated settlement for 25% annular space collapse with 2.5% ground loss, is 10.5 mm, which is below the Level 1 “Alert - (Review Threshold, 11 mm)” and Level 2 “Critical - (Stop Work, 22 mm)” settlement limits for the CP Class 2 track.

It is the contractor’s responsibility to maintain the required annular space collapse and ground loss to adhere to the settlement limits for the CP Class 2 track. It is the contractor’s responsibility to determine the means and methods for the steel casing installation. Based on AECOM’s local experience, the clays remold shortly after installation, reducing annular space/voids. If excessive settlements are observed over time, post-annular grouting or other mitigation measures should be implemented. The maximum radial overcut (25.4 mm) shall be communicated to the contractor in the technical specifications for the casing installation.

The potential for ground surface movement depends on the contractor’s work methods, equipment, and techniques (e.g., soil plug, guided auger). A settlement monitoring plan will be implemented to monitor settlement during installation (pre and post-construction) and adjust the methodology as needed before reaching the crossing location. Section 8 further discusses the settlement monitoring plan in accordance with CPR Geotechnical Protocol for Pipeline and Utility Crossings Under Railway Tracks.

Section 8.4 discusses the settlement monitoring program in accordance with CPR Geotechnical Protocol for Pipeline and Utility Crossings Under Railway Tracks.

8. Track Settlement Monitoring Plan

CP provides the requirements for settlement monitoring on “Geotechnical Protocol for Pipeline and Utility Crossing(s) Under Railway Tracks” document dated May 15, 2024. A copy of this document is included in **Appendix III** for reference. This document includes the minimum monitoring frequency requirements for preconstruction, during construction and post-construction. It also identifies two alarm levels to be implemented during the settlement monitoring.

8.1 Monitoring Point Layout

The location of the settlement monitoring points is illustrated on Drawing No. 2 in **Appendix II**.

The installation of the monitoring points shall be as follows:

- As per section 9.2.2 of the *CPKC Geotechnical Protocol for Pipeline and Utility Crossing(s) under Railway Tracks*, first set of sub-surface monitoring points to be placed on either side of the outside rail at 2m distance off track centerline measured from outside of the rails. Additional sub-surface points to be placed at the toed of slope and at end points/toes of ZPTL. Signal and fibre locates to be completed before installing any settlement monitoring equipment in the railway right of way.
- Surface (rail) monitoring points will be installed along each side of the BOR (east and west). These points will be placed directly at the base of both rails, spaced 9.45 m apart, over the projected settlement trough. This setup will monitor the differential transversal elevation between the rails. AECOM recommends a total of 22 surface monitoring points, centered on each carrier pipe alignment.
- The subsurface settlement points will be installed 1 m above the crown of the casing. These points will be installed at an elevation of 228.49 m. AECOM recommends 6 sub-surface monitoring points. The upper portion of the iron bar will be d in PVC piping and backfilled with bentonite to prevent water infiltration, and the lower portion will be filled with sand. The installation will include a cover at the ground surface to protect against disturbance (typically a flush-mount or stick-up well casing).

The typical installation and decommissioning details for the surface and sub-surface in-ground settlement monitoring points are shown on pages 36 and 37 of the CP Geotechnical Protocol for Pipeline and Utility Crossing(s) Under Railway Tracks as shown in **Appendix III**.

8.2 Settlement Monitoring Frequency

Track Movement Monitoring Guidelines for Trenchless Pipe Installation of the CP Geotechnical Protocol describes the minimum required frequency of the settlement monitoring points at various times. The subsurface settlement points will be monitored simultaneously with the surface settlement points which act as a precursor to potential surface movement during pipe installation. All monitoring points will be surveyed to the typical industry standard accuracy of +/- 2 mm. In accordance with CP’s monitoring guidelines, a monitoring program of all points is to be conducted once the installation is complete. The instructions listed are to be followed:

- To avoid real time monitoring, the contractor shall complete the crossing under the ZPTL in one day. If the crossing is not completed under the ZPTL in one day, the contractor may be requested to provide real time monitoring overnight due to CPKC not being able to provide flagging services overnight.
- **Pre-construction.** Monitoring will start before the excavation of the pits and pipe installation begins and readings should be taken twice per day for no less than two days. This is required to establish a reliable methodology and demonstrate the accuracy to be achieved. The collection of the baseline readings will be done by the surveyors. AECOM surveyor will collect two (2) baseline readings daily on two (2) consecutive days. A memo will be prepared summarizing the baseline readings.

- **During Construction.** Monitoring will proceed through the construction period and will be completed twice (2) daily (for branch lines/line with low traffic Class 1-2 Track). This will be in coordination with the site surveyor. Daily reports will be prepared to include all settlement monitoring data, along with pertinent photos.
- **Post Construction.** Monitoring will continue three (3) days after completion of the construction. A memo will be prepared summarizing the monitoring points.
- If there is any loss of ground during pipe installation, any reason to believe settlement may be delayed or any settlement is identified during the installation of pipe or subsequent monitoring period, the monitoring will be continued until AECOM's Geotechnical Engineer of Record (GER) deems it is safe to discontinue such monitoring.

In accordance with CP Protocol, the GER, German Leal, M.Eng., P.Eng. will provide a final sealed and stamped final report to CP approved service provider with a copy to CP Public Works – Utilities supervisor confirming that the work has been completed in accordance with the approved plans and procedures. In addition, the GER will collaborate with an AECOM geotechnical engineer (experienced with CP crossings), Sonny Chang, M.Sc., P.Eng. to provide additional support for the surface and sub-surface settlement monitoring program.

8.3 Ground Movement Alarm Level

AECOM adopts the following criteria for the settlement monitoring. This criterion is applicable to both the surface and subsurface monitoring points and is based upon Class 2 track. According to the *Track Movement Monitoring Guidelines for Trenchless Pipe Installation* from **Appendix III** of the CP Geotechnical Protocol, there are two alarm levels for ground movement. The two alarm levels are as follows:

- Level 1: "ALERT - (REVIEW THRESHOLD)": maximum value of 11 mm.
- Level 2: "CRITICAL - (STOP WORK)": a value ≥ 22 mm.

8.4 Settlement Monitoring Program

CP requires carrying out track settlement monitoring (i.e., surface and subsurface settlement points) before, during and after construction. The intent of subsurface settlement points is to measure soil settlement, if any, above the pipe during construction in order to predict the potential movement of the tracks above.

8.4.1 Pre-construction Tasks

Prior to commencement of construction AECOM will complete the following tasks:

- Submit the scope of the proposed settlement monitoring program to City of Winnipeg to include in the permit submission to CP.
- Review and incorporate any comments received from CP into the scope of the proposed settlement monitoring program.

On the receipt of the CP permit from City of Winnipeg, AECOM will:

- Prior to construction, locate any utilities and/or fiber optics within the ROW. This will be the contractor's responsibility.
- Prepare the monitoring point installation such as underground clearance and Click-Before-You-Dig MB. It is understood that utility locates will be the contractor's responsibility. The subsurface monitoring points will be installed by the contractor under the supervision of AECOM geotechnical personnel. The utility locates program as follows:
    - Public utility locates for the area of the proposed monitoring points;
    - Submit to CP to obtain utility locates specific to CP railway operations; and

  - Retain private utility locate company to identify and mark services and/or utilities in accordance with AECOM's standard ground disturbance protocol.
- Identify if any utilities/services are within 3 m of the intended locations of the monitoring points and if so, said utilities/services will require positive identification (hand or hydro-vac exposure in the field). Where positive exposure is not practical, the locations of the proposed monitoring points will be adjusted accordingly. This will be contractor's responsibility.
- Develop an emergency contact list that identifies representatives from AECOM, City of Winnipeg, CP, the pipe installation contractor, and additional parties as may be identified at that time. The emergency contact list will be distributed to all parties to be used in the event that the settlement Alert - (Review Threshold) Limit range is exceeded.
  - An emergency contact list of CP personnel will be prepared by AECOM and distributed to all applicable parties once the CP crossing permit has been obtained by the City of Winnipeg.
  - The purpose of the emergency contact list is to notify CP representatives of excessive or unexpected settlement during construction. Coordinate with the project surveyor to layout the proposed alignment of the installation between jacking and receiving locations and to layout the locations of the planned surface and sub-surface monitoring points.
- Contact the railway operator to request a Protective Person (flagman) and coordinate access to enter the ROW for the described work. This will be contractor's responsibility. The contractor shall provide CPKC with the intended contract working hours when submitting for the CPKC crossing agreement and flagging application. This is to allow CPKC to conduct their internal approvals if overtime for flaggers is required.

On the completion if the preceding tasks and receipt of approval from CP to proceed, AECOM will:

- Arrange a pre-construction meeting with all stakeholders to discuss project and construction details including work description, construction methods and schedule, restrictions, safety, work duration, daily reporting, and other CP requirements.
- Submit to the railway for a Protective Person (flagman) to be onsite and to coordinate access to enter the ROW. This will be contractor's responsibility.
- Oversee the installation of the monitoring points as outlined in the proposed settlement monitoring program with adjustments as required for the presence of utilities/services. There will be six (6) sub-surface points installed. A drill rig will be used to install the subsurface monitoring points.
- AECOM surveyor will collect two (2) baseline readings daily on two (2) consecutive days.
- Collection of baseline readings on all monitoring points will be within 1 – 2 weeks of the commencement of construction. This timeline can be revised at the discretion of the geotechnical consultant subject to the prevailing subsurface conditions, activity in the area (construction or otherwise), and climate/weather conditions during the period prior to and leading up to the commencement of construction.
- A memo will be prepared summarizing the baseline readings. The memo will be submitted within 5 days upon completion of the baseline readings.

8.4.2 Construction Tasks

Monitoring will proceed through the construction period and will be completed twice (2) daily (for branch lines/line with low traffic Class 1-2 Track). This will be in coordination with the site surveyor. Daily reports will be prepared to include all settlement monitoring data, along with pertinent photos. The GER (German Leal) will conduct a site visit once per week to oversee the surface and sub-surface settlement monitoring program. The GER will communicate with the AECOM geotechnical engineer to provide updates on the settlement monitoring.

If the results of the survey are above the Level 1: "Alert - (Review Threshold)" and Level 2: "Critical - (Stop Work)". The following will be implemented:

8.4.2.1 Level 1: "ALERT - (REVIEW THRESHOLD)"

If the measured subsurface and/or surface settlement points are above the Alert - (Review Threshold) Level (i.e., more than 11 mm):

- Notify all parties on the emergency contact list within 24 hours that the results of the monitoring are within the Alert - (Review Threshold) Level.
- A survey of the surface point will be carried out and work will be authorized to continue if no movement of the subsurface point has been measured from the previous reading. In this case, request that the Project Surveyor undertake an additional survey to confirm the results obtained and provide a verbal report of the results to the geotechnical consultant within 1-hour of completion of the survey and a written report of the results to the geotechnical consultant within 24 hours.
- Notify all parties on the emergency contact list within 24 hours of the results of the additional monitoring.

8.4.2.2 Level 2: "CRITICAL - (STOP WORK)"

If the measured subsurface and/or surface settlement points are within a Critical - (Stop Work) Level (i.e., more than 22 mm):

- Mobilize geotechnical staff to the Site within 12 hours to identify if there are any obvious visual indications of movement of the rail tracks, rail ballast, rail embankment or similar and/or if there is any indication of the development of ground subsidence, sink holes or slope instability.
- Notify all parties on the emergency contact list, including CP, immediately that the monitoring results are above the Critical - (Stop Work) Level.
- Communicate with project team, who shall advise the contractor to cease the drilling operations immediately until an assessment of the observed settlement is conducted by a geotechnical engineer and a conference call/meeting is convened between CP, City of Winnipeg, the contractor and the Geotechnical Consultant to discuss the results of the assessment.
- A survey of the surface points will be carried out and work will be authorized to continue if no movement is measured for at least two (2) readings taken 12 hours apart. In this case, request that the project surveyor undertake an additional survey to confirm the results obtained and provide verbal report of the results to the Geotechnical Consultant immediately on completion of the survey and a written report of the results to the Geotechnical Consultant within 24 hours.
- Notify all parties on the emergency contact list within 24 hours of the results of the additional monitoring and the results of the visual observations of the current conditions.

8.4.3 Post Construction Task

Decommissioning of the monitoring points will begin once post-construction monitoring has been completed (twice daily for three (3) consecutive days), and all parties have reviewed the monitoring data and are satisfied that, if any movement was detected during the monitoring period (if any) have stopped.

For the sub-surface monitoring points, the protective covers will be removed, the iron bars will be removed from CP ROW. The sub-surface monitoring points will be backfilled with bentonite pellets. All backfill material from the installation of the monitoring points will be removed from CP right of way as well.

The site shall be restored to its original condition within the CPKC ROW which includes decommissioning of surface and subsurface monitoring points.

A final memo and as-built drawings will be submitted at the end of the project. The memo will summarize the settlement monitoring that was performed for the LDS crossing installation and confirms that the work was completed in general accordance with the submitted plans and procedures. The final memo and the as-built drawings will be sealed and stamped by the GER, German Leal, M.Eng., P.Eng.

9. Conclusions

In general, and based on the available information, it is recommended that the proposed LDS be installed using trenchless methodologies. A pipe jacking system utilizing either guided auger boring or guided pipe ramming should be adopted. These methods are deemed appropriate given the required installation parameters and based upon the subsurface ground and groundwater conditions. It is the trenchless contractor's responsibility to select a suitable trenchless method based on their means and methods, local experience and trenchless equipment.

Characterization of groundwater conditions may be required to validate dewatering quantities, methodologies and techniques prior to the onset of construction.

Given the possibility that installation will occur within the fat clay layer, the contractor should be prepared to mitigate against instability at the face of the bore path as described in this Report. The contractor should submit a construction methodology, including mitigation techniques for adverse track settlement, to the engineer for approval prior to installation. The contractor shall submit a recovery plan, outlining the steps to be implemented in the event of failure (e.g., excessive ground loss or settlement/collapse, heaving, etc.), to the GER. A reviewed copy of this plan shall then be provided to CPKC prior to construction. Throughout the pipe installation process, surface monitoring should be undertaken to evaluate the impact of pipe jacking/pipe ramming beneath the CP rail tracks. Should observed surface settlement and heave values exceed the maximum anticipated values, the Contractor should implement the noted action plan to correct unwanted settlement.

Appendix I

NEWPCC Upgrade: Biosolids Facilities Early Works Geotechnical Data Report

NEWPCC Upgrade: Biosolids Facilities Early Works

Geotechnical Data Report
Land Drainage System Crossing

WSTP

60705950

October 2024

Delivering a better world

AECOM

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Lana Obach, P.Eng., M.A.Sc., PMP
Project Manager
City of Winnipeg
Unit 110, 1199 Pacific Avenue
Winnipeg, MB R3E 3S8

October 28, 2024

Project #
60705950

Subject: NEWPCC Upgrade: Biosolids Facilities Early Works – Geotechnical Data Report

Dear Ms. Obach:

AECOM Canada Ltd. is please to submit our Geotechnical Data Report for the above referenced project.

Should you have any queries, please contact German Leal directly at (431) 335-9734.

Sincerely,
AECOM Canada Ltd.

German Leal, M.Eng., P.Eng.
Geotechnical Discipline Lead
german.leal@aecom.com

GL:ag
Encl.

Statement of Qualifications and Limitations

The attached Report (the "Report") has been prepared by AECOM Canada Ltd. ("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").

The information, data, recommendations and conclusions contained in the Report (collectively, the "Information"):

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- represents AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to AECOM which has not been independently verified;
- has not been updated since the date of issuance of the Report and its accuracy is limited to the time period and circumstances in which it was collected, processed, made or issued;
- must be read as a whole and sections thereof should not be read out of such context;
- was prepared for the specific purposes described in the Report and the Agreement; and
- in the case of subsurface, environmental or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time.

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AECOM: 2015-04-13

Quality Information

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Revision History

Rev #	Revision Date	Revised By:	Revision Description
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Executive Summary

AECOM Canada Ltd. (AECOM) was retained by the Winnipeg Sewage Treatment Program (WSTP) to provide geotechnical services to support the design and construction of the land drainage sewer (LDS) for the North End Water Pollution Control Centre (NEWPCC). The proposed LDS will be crossing the Canadian Pacific (CP) railway and will be connected to the future Parcel B system.

As part of the NEWPCC Upgrades: Biosolids Facilities Early Works, AECOM was tasked with completing a geotechnical field investigation, a laboratory testing program, and a summary of the findings of geotechnical investigation.

The geotechnical investigation program consisted of a subsurface exploration program (drilling, soil sampling and in-situ tests). A total of two testholes (TH24-02 and TH24-08) were drilled: one testhole drilled east of CP railway and one testhole drilled west of CP railway.

The soils encountered during the investigation consisted of topsoil, fill (clay fill), lean clay, fat clay and silt till. Selected representative soil samples were tested to determine physical characteristics, evaluate the engineering properties, and aid with further characterization of the subsurface.

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Tables

Appendices

1. Introduction

The Winnipeg Sewage Treatment Program (WSTP) has retained AECOM Canada Ltd. (AECOM) to provide geotechnical engineering services to support the design and construction of the land drainage sewer (LDS) as part of the North End Water Pollution Control Centre (NEWPCC) Upgrades: Biosolids Facilities Early Works. The proposed LDS will be crossing the Canada Pacific (CP) railway connecting the future Parcel B system to a new storm retention basin (SRB).

This report documents the findings of the geotechnical investigation for the CP crossing completed near the NEWPCC in Winnipeg, Manitoba and characterizes the subsurface and groundwater conditions. A plan view of the site investigation extent is shown in **Appendix A**.

1.1 Project Site and Proposed Construction

The project site is located near the existing NEWPCC in Winnipeg, Manitoba. The proposed construction and design for the LDS will include a reinforced concrete pipe in the future Chief Peguis Trail right-of-way.

This report addresses the findings of the geotechnical investigation for the LDS segment that will be crossing the CP railway and focuses on the two testholes drilled east and west of the CP railway.

2. Geotechnical Investigation

2.1 Drilling and Sampling Program

The subsurface drilling and sampling program was conducted on October 13, 2023 and December 15, 2023. The location of public utilities at the site were provided by Click Before You Dig Manitoba and DigShaw, while a final complete utility locate was identified and marked by a private locator. Drilling program was completed by Paddock Drilling Ltd. under the supervision of AECOM's geotechnical field personnel. The field investigation consisted of drilling two (2) testholes to support the design and construction of the LDS crossing. One testhole was drilled east of the CP railway (in Parcel A) and one testholes was drilled west of the CP railway (in Parcel B). The testholes were drilled to depths ranging from 10.7 m to 17.1 m: one testhole was terminated in the fat clay and one testhole was terminated up to auger refusal at very dense till.

Subsurface conditions observed during testhole drilling were visually observed and documented by AECOM geotechnical personnel according to the Unified Soil Classification System (USCS). Representative soil samples were obtained directly from the auger flights at 0.3 m to 1.5 m intervals. The undrained shear strength of cohesive soils was evaluated using a field torvane. Four (4) undisturbed samples were retrieved in thin-walled Shelby tubes. Split spoon sample was collected from the underlying silt till material to obtain Standard Penetration Test (SPT) "N" values. Groundwater, seepage and sloughing conditions were recorded upon completion of drilling. The testholes were backfilled with auger cuttings and bentonite chips.

Testhole logs have been prepared for each testhole to record the descriptions and the relative positions of the soil strata, locations of samples obtained, laboratory test results and other pertinent information. Soil profiles have been prepared for representative sections. The testhole logs and soil profiles are included in **Appendix B**.

The testhole locations drilled during geotechnical investigation is summarized in **Table 2-1**.

Table 2-1: Summary of the Testholes Drilled – Subsurface Investigation

Testhole I.D.	Location	Coordinates	Ground Elevation (m)	Completion Depth (m)	Termination USCS Soil Type
TH23-02	Parcel A – East of CP railway	5535408 m N, 635420 m E	230.43	10.7	Fat Clay
TH23-08	Parcel B – West of CP railway	5535448 m N, 635349 m E	230.29	17.1	Silt Till

2.2 Instrumentation

During the geotechnical field investigation, one (1) standpipe piezometers (SP) consisting of 25 mm in diameter and 0.9 m in length screening Casagrande tip was installed. The installation details of the standpipe piezometers are shown on the testhole logs in **Appendix B** and summarized in **Table 2-2**.

Table 2-2: Standpipe Piezometer Installed for GWL Reading

Testhole I.D.	SP depth (m)	Tip Elevation (m)	Slotted Layer USCS Soil Type
TH23-08 (SP1)	17.0	213.43	Silt Till

3. Laboratory Testing Program

The laboratory testing program was developed to measure the index properties of the different soil types encountered. The laboratory tests consisted of geotechnical testing on disturbed grab, split spoon samples and undisturbed Shelby tubes samples. The geotechnical tests were conducted at AECOM's Materials Testing Laboratory in Winnipeg, Manitoba. A summary of the tests performed is presented below and detailed laboratory results are presented in **Appendix C**.

3.1 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 determination of moisture content, Atterberg Limits, grain size distribution by hydrometer method and unconfined compressive strength on samples collected during the field investigation. A summary of the geotechnical testing that was completed is provided in **Table 3-1**.

Table 3-1: Summary of Laboratory Testing

Laboratory Test	Number of Tests	Testing Standard
Moisture Content	29	ASTM D2216
Grain Size Distribution (Hydrometer Analysis)	7	ASTM D422
Atterberg Limits	7	ASTM D4318
Unconfined Compressive Strength	3	ASTM D2850

4. 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.

4.1 Subsurface Profile

Soils encountered during the investigation consisted of the following:

- Topsoil
- Fill (Clay fill)
- Lean clay
- Fat clay
- Silt till

The description of the subsurface soil units encountered at east and west side of the CP crossing is provided in the following subsections. The detailed descriptions of the subsurface conditions are provided in the testhole logs in **Appendix B**, and the laboratory lab results are provided in **Appendix C**.

4.1.1 Topsoil

Topsoil was encountered at the ground surface of the testholes. The topsoil extended to depths ranging from 0.3 m to 0.5 m. The moisture content ranged from 41.9% to 55.5% with an average of 48.7%.

4.1.2 Fill

Fill (Clay fill) was encountered below the topsoil in testhole TH23-08. The fill extended to a depth of 1.1 m. The undrained shear strength of the fill was 69 kPa, classifying the material as stiff in consistency. The fill was black in colour, was clay with silt, contained some sand and trace gravel. The moisture content of the fill was 42.4%.

4.1.3 Fat Clay (CH)

Fat clay was encountered below the topsoil in TH23-02 and below fill in testhole TH23-08. The fat clay was encountered at depths ranging from 0.5 m to 1.1 m and was extended to depths ranging from 1.1 m to 1.5. Then, another layer of fat clay was encountered below the lean clay at depths ranging from 2.3 m to 3.0 m and extended to a depth of 16.2 m. It was brown in colour changing to grey below 6.1 m. It was high in plasticity, was silty, contained traces of sand and gravel. The undrained shear strength of the fat clay ranged from 9.8 kPa to 68.7 kPa (average of 33.86 kPa), generally decreasing with depth, classifying the material as stiff to soft in consistency. The moisture content of the fat clay ranged from 31.10% to 61.70% with an average of 47.31%.

4.1.4 Lean Clay (CL)

Lean clay was encountered below the first layer of fat clay in all testholes. The lean clay was encountered depths ranging from 1.5 m to 2.0 m and was extended to depths ranging from 2.3 m to 3.0 m. It was tan in colour, soft in consistency, low in plasticity, contained some clay and traces of sand. The moisture content of the lean clay ranged from 23.60% to 25.20% with an average of 24.47%.

It should be noted that silt content of lean clay based on the results of grain size analysis ranged between 79.8% and 84.5% indicating the lean clay has high silt content. (Refer to **Table 6-1** of Section 6).

4.1.5 Silt Till (SP)

Glacial silt till was encountered below the fat clay in testhole TH23-08 encountered at a depth of 16.15 m, and extended to auger refusal at a depth of 17.1 m. The silt till was tan in colour, was clayey, and contained traces gravel. Standard Penetration Tests (SPT) completed within the silt till show uncorrected “N” values of 50+ blows per 300 mm of penetration, classifying the material as very dense in relative density. Generally, the sandy silt till increases in relative density with increasing depth. The moisture content of the sandy silt till ranged from 54.0% to 43.2% with an average of 48.6%. Although not encountered during drilling, cobbles and boulders are commonly found within the sandy silt till.

5. Groundwater and Sloughing Conditions

Groundwater seepage and sloughing conditions were recorded upon completion of drilling of each testholes. Details of the location and nature of the sloughing and seepage conditions, as well as conditions of the groundwater encountered are provided on the testhole logs in **Appendix B** and presented in **Table 5-1**.

Table 5-1: Summary of Observed Groundwater Seepage and Sloughing Conditions

Testhole I.D.	Groundwater Seepage	Observed Depth of Groundwater Seepage (m)	Depth of Groundwater Upon Completion of Drilling (m)	Observed Depth of Soil Sloughing (m)
TH23-02	Minor	1.5	7.5	None
TH23-08	Moderate	2.0	None	13.7

Only short-term seepage and sloughing conditions were observed. It should be noted that groundwater levels (GWL) and subsequently the seepage and sloughing depths may change seasonally, annually or as a result of construction activities.

5.1 Standpipe Piezometer Monitoring Result

One standpipe piezometer (SP) was installed in testhole TH23-08 slotted within the silt till layer to monitor and measure the groundwater level in the testhole.

Groundwater depth was measured within the standpipe. The measured groundwater depth and elevation with corresponding date are provided in **Table 5-2**.

Table 5-2 : Groundwater Readings

Parameters	TH23-08 (SP1)
Testhole Elevation (m)	230.29
Tip Depth (m)	17.0
Tip Elevation (m)	213.43
USCS Soil Type at Tip Location	Silt Till
Dates	**GWL Depth Below Ground Surface (m)**
January 24, 2024	1.38 (Elev. 228.91 m)
January 26, 2024	1.42 (Elev. 228.87 m)
October 23, 2024	1.62 (Elev. 228.68 m)

It should be noted that the groundwater elevations read from the SP installed in TH23-08 are higher than the groundwater elevation encountered within the study limits and may not be a representative of the groundwater elevation. Two other standpipe piezometers were installed within the study limit (included in a separate report) slotted within the silt till layer. The groundwater elevation within these standpipe piezometers ranged between Elev. 224.75 m and Elev. 225.21 m.

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 leave clay (i.e., with high silt content) and silt till layer.

6. Laboratory Test Results

The results of the laboratory tests are presented in tables within this section, and the laboratory test reports are provided in **Appendix C**.

Table 6-1: Grain Size Distribution (Hydrometer Analysis) Results

Testhole No.	Sample ID	Sample Depth (m)	Grain Size Distribution (%)			
			Gravel 75 to 4.75 mm	Sand <4.75 to 0.075 mm	Silt <0.075 to 0.002 mm	Clay <0.002 mm
TH23-02	G2	0.6 – 0.8	0.0	0.3	21.9	77.8
	G4	2.1 – 2.3	0.0	7.6	79.8	12.6
	G8	5.9 – 6.1	0.0	0.4	25.0	74.6
TH23-08	T4	1.5 – 2.0	0.0	5.3	45.6	49.1
	G5	2.1 – 2.3	0.0	7.6	84.5	7.9
	T11	7.6 – 8.1	0.0	1.5	38.4	60.2
	G12	9.0 – 9.1	1.5	10.8	39.9	47.7

Table 6-2: Atterberg Limit Test Results

Testhole No.	Sample ID	Sample Depth (m)	USCS Soil Type	Liquid Limit	Plastic Limit	Plasticity Index	Activity
TH23-02	G2	0.6 – 0.8	CH	93	27	66	0.85
	G4	2.1 – 2.3	CL	25	15	10	0.79
	G8	5.9 – 6.1	CH	91	25	66	0.88
TH23-08	T4	1.5 – 2.0	CH	69	19	51	1.04
	G5	2.1 – 2.3	CL	23	12	11	1.39
	T11	7.6 – 8.1	CH	80	21	59	0.98
	G12	9.0 – 9.1	CH	73	17	56	1.17

Table 6-3: Unconfined Compressive Strength Test Results

Testhole No.	Sample ID	Sample Depth (m)	Soil Type	Moisture Content (%)	Bulk Unit Weight (kN/m^3)	Undrained Shear Strength (kPa)	Unconfined Compressive Strength (kPa)
TH23-08	T4	1.5 – 2.0	Clay	35.6	19.4	23.2	46.5
	T8	4.6 – 5.0	Clay	53.4	17.0	16.3	32.6
	T11	7.6 – 8.1	Clay	46.2	17.3	33.2	66.5

7. Frost

7.1 Seasonal Frost Penetration

The depths of frost penetration have been estimated for a range of annual air freezing identified in **Table 7-1**.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). 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 7-1: 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)	2.1	2.2	2.4
Estimated Frost Penetration (Fat Clay Subgrade) – grass with snow cover (m)	2.0	2.0	2.3

7.2 Frost Susceptivity

The qualitative frost susceptibility of a soil is typically assessed using guidelines developed by Casagrande (1932) on the basis of the percentage by weight of the soil finer than 0.02 mm, and the Plasticity Index. This 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 (fat clay, lean clay and silt) encountered during the geotechnical investigation mostly within the frost group 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 7-2**.

Table 7-2: Project Site Frost Susceptibility

Soil Unit	USCS Soil Type	Frost Group	Percentage finer than 0.02 mm, by weight	PI	Frost Susceptibility
Fat Clay	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*

8. 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.

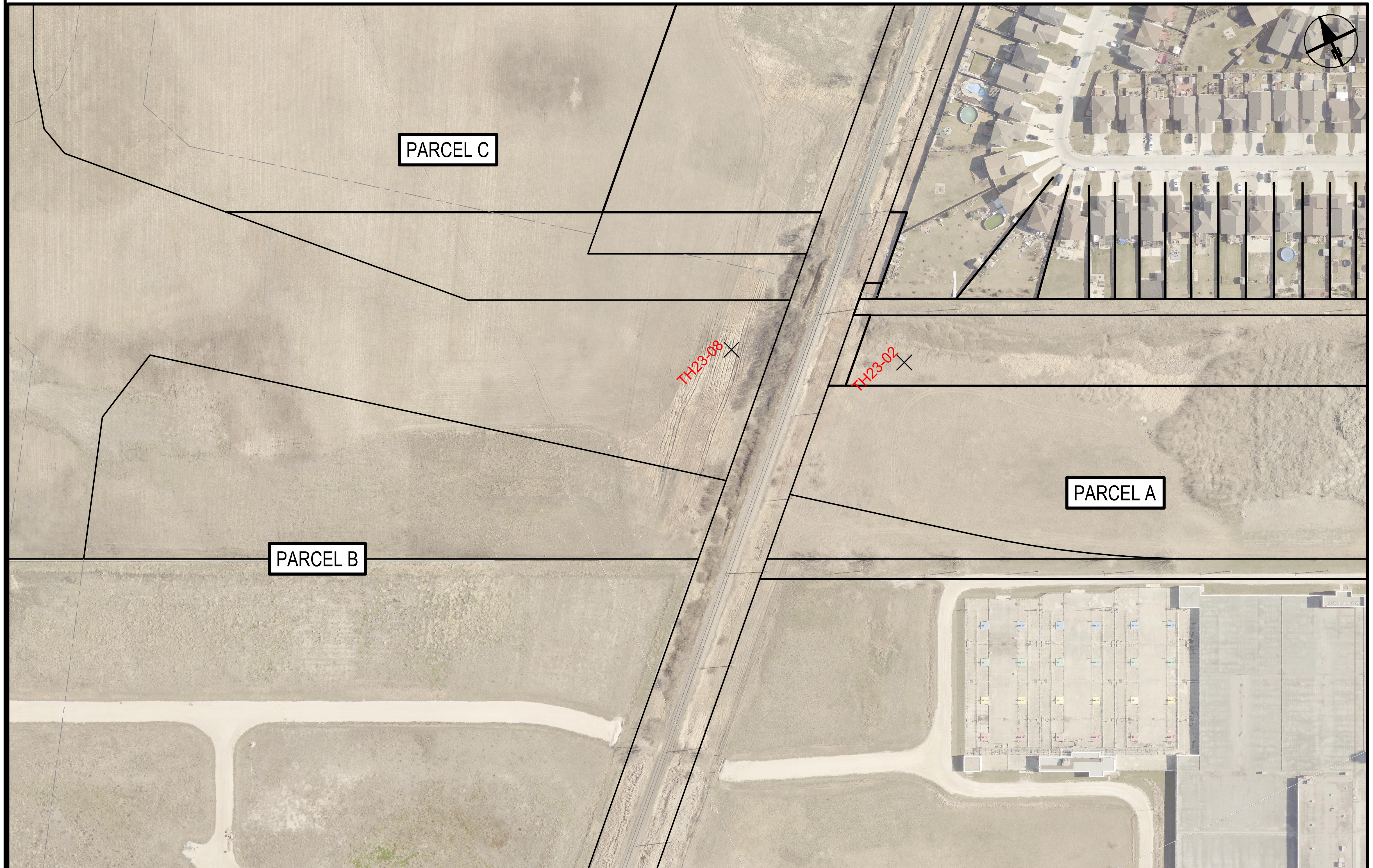
The 2020 NBCC Seismic Hazard Calculation for the site is provided in **Appendix D.** It includes values of spectral acceleration (for time periods of 0.05, 0.1, 0.2, 0.3, 0.5, 1.0, 2.0, 5.0 and 10.0 seconds), peak ground acceleration, and peak ground velocity for 2%, 5%, 10% and 40% probability of exceedance in 50 years.

Appendix A

Testhole Location

Point Table

Northing	Easting	Description
5535448.38	635348.76	TH23-08
5535408.30	635419.85	TH23-02



0 50 100 m

1:2000

Winnipeg

AECOM

Winnipeg Sewage Treatment Program
NEWPCC Upgrade Project

TESTHOLE LOCATION PLAN
Figure: 1

Appendix B

Testhole Logs

PROJECT: NEWPCC Biosolids LDS Crossing	CLIENT: WSTP	TESTHOLE NO: **TH23-02**
LOCATION: NEWPCC, Winnipeg, MB UTM Zone 14U: 5535408.134 m N, 635419.979 m E		PROJECT NO.: 60705950
CONTRACTOR: Paddock Drilling	METHOD: Solid Stem Auger	ELEVATION (m): 230.43

SAMPLE TYPE: GRAB | SHELBY TUBE | SPLIT SPOON | BULK | NO RECOVERY | CORE

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

DEPTH (m)	USC	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE #	SPT (N)	COMMENTS	ELEVATION
0	OR	Topsoil	GRAB	G1			230
	CH	Stiff brown fat CLAY (CH)	GRAB	G2		G2: LL = 93, PL = 27, PI = 66, Gravel = 0%, Sand = 0.3%, Silt = 21.9%, Clay = 77.8%	
1			GRAB	G3			229
2	CL	Soft tan lean CLAY (CL)	GRAB	G4		G4: LL = 25, PL = 15, PI = 10, Gravel = 0%, Sand = 7.6%, Silt = 79.8%, Clay = 12.6%	228
3	CH	Firm brown fat CLAY (CH)	GRAB	G5			227
4			GRAB	G6			226
			GRAB	G7			
5							225
6		- Grey below 6.1 m	GRAB	G8		G8: LL = 91, PL = 25, PI = 66, Gravel = 0%, Sand = 0.4%, Silt = 25%, Clay = 74.6%	224
7							223
8			GRAB	G9			222
9			GRAB	G10			221
10							220
		- Soft below 10.5 m	GRAB	G11			

END OF TESTHOLE
- Testhole terminated at a depth of 10.7 m in fat CLAY
- Minor groundwater seepage was observed at a depth of 1.5 m.
- Groundwater level was observed at a depth of 7.5 m upon completion of drilling.
- No soil sloughing was observed during or upon completion of drilling.
- Testhole backfilled with soil cuttings and bentonite.

Depth scale: 0–25 m; Elevation scale: 230–206 m

AECOM

LOGGED BY: AB	COMPLETION DEPTH: 10.67 m
REVIEWED BY: GL	COMPLETION DATE: 10/23/23
PROJECT ENGINEER: German Leal	Page 1 of 1

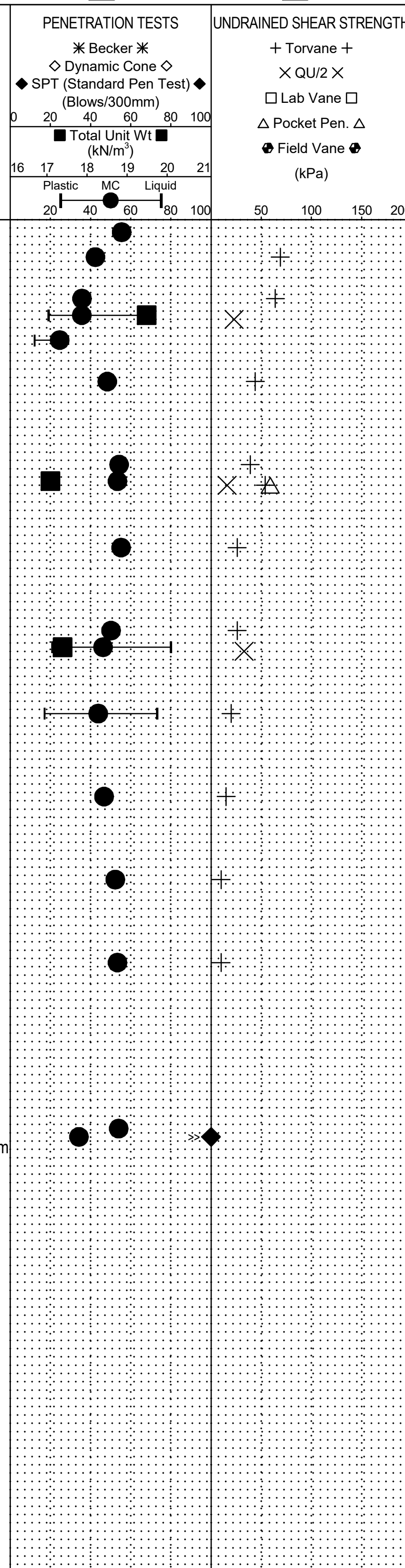
PROJECT: NEWPCC Biosolids LDS Crossing	CLIENT: WSTP	TESTHOLE NO: **TH23-08**
LOCATION: NEWPCC, Winnipeg, MB UTM Zone 14U: 5535448.050 m N, 635348.646 m E		PROJECT NO.: 60705950
CONTRACTOR: Paddock Drilling	METHOD: Solid Stem Auger	ELEVATION (m): 230.29

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	G1		
	FILL	FILL: brown silty clay, trace gravel, trace sand	G2		
1	CH	Stiff brown fat CLAY (CH)	G3, T4		T4: LL = 69, PL = 19, PI = 51, Gravel = 0%, Sand = 5.3%, Silt = 45.6%, Clay = 49.1%
2	CL	Soft tan lean CLAY (CL)	G5		G5: LL = 23, PL = 12, PI = 11, Gravel = 0%, Sand = 7.6%, Silt = 84.5%, Clay = 7.9%
2.3	CH	Firm brown fat CLAY (CH)	G6, G7, T8, G9		
		- Grey below 6.1 m	G10, T11		T11: LL = 80, PL = 21, PI = 59, Gravel = 0%, Sand = 1.5%, Silt = 38.4%, Clay = 60.2%
			G12		G12: LL = 73, PL = 17, PI = 56, Gravel = 1.5%, Sand = 10.8%, Silt = 39.9%, Clay = 47.7%
		- Soft below 10.5 m	G13, T14, G15, G16		
16.2	TILL	Very dense brown TILL (SP)	G17, S18	200/ 152mm	

END OF TESTHOLE
- Testhole terminated at a depth of 17.1 m at auger refusal on very dense TILL
- Moderate groundwater seepage was observed at a depth of 2.0 m.
- Soil sloughing was observed in fat CLAY at a depth of 13.7 m.
- Standpipe piezometer installed to a depth of 17.0 m, in TILL slotted between a depth of 16.2 m and 16.8 m, stick up 0.3 m.
- Testhole backfilled with soil cuttings and bentonite.



ELEVATION: 230, 229, 228, 227, 226, 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213, 212, 211, 210, 209, 208, 207, 206

AECOM	LOGGED BY: AB	COMPLETION DEPTH: 17.07 m
	REVIEWED BY: GL	COMPLETION DATE: 12/15/23
	PROJECT ENGINEER: German Leal	Page 1 of 1

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

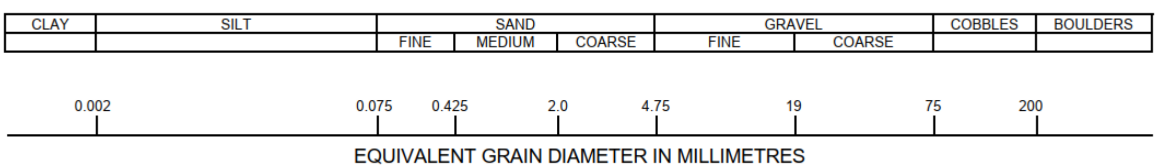
1.3 Grian Size Distrubtion

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:

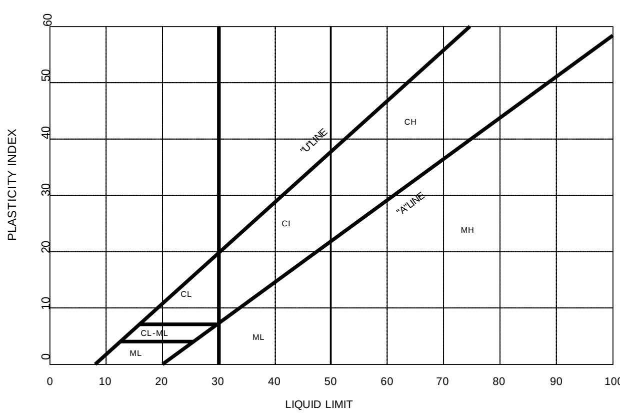
Table 1 Cohesive Soils

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) PASSING	SIEVE SIZE (mm) RETAINED	DEFINING RANGES OF PERCENTAGE BY WEIGHT OF MINOR COMPONENTS 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

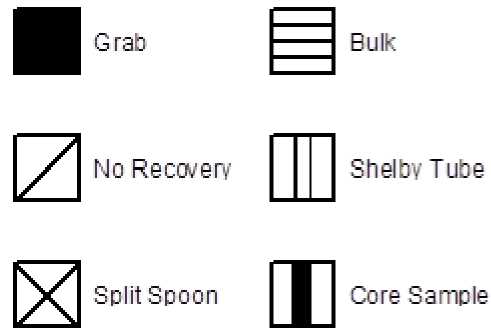
February 2022

1.5 Sample Type, Symbols and Abbreviations

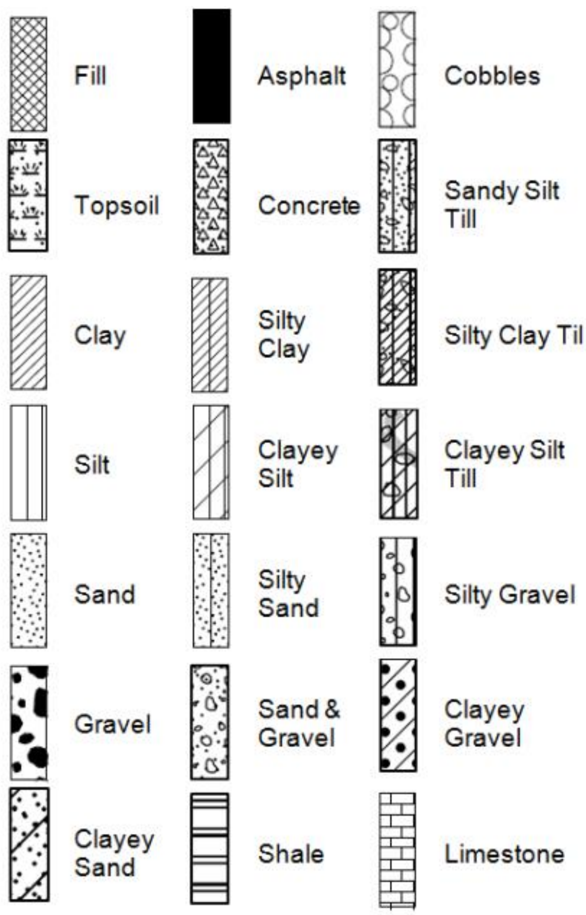
The depth, type, and condition of samples are indicated on the logs by the following symbols or abbreviations:

Sample abbreviations:
GS: Grab 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

Symbols:



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



AECOM

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*

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, % At 6 months	At 12 months††	Maximum expansion when tested using CSA A3004-C8 Procedure B at 5 °C, % ††† 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 (▼).

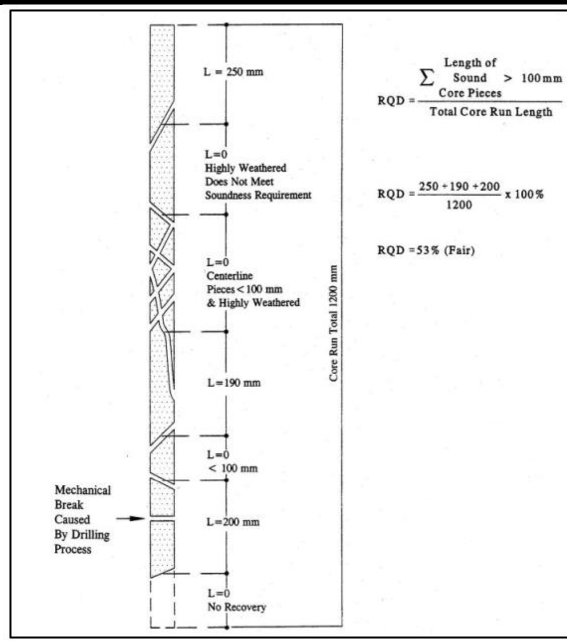
4. EXPLANATION OF ROCK

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	0 – 25 Very Poor Quality	
25 – 50 Poor Quality	25 – 50 Poor Quality	
50 – 75 Fair Quality	50 – 75 Fair Quality	
75 – 90 Good Quality	75 – 90 Good Quality	
90 – 100 Excellent 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

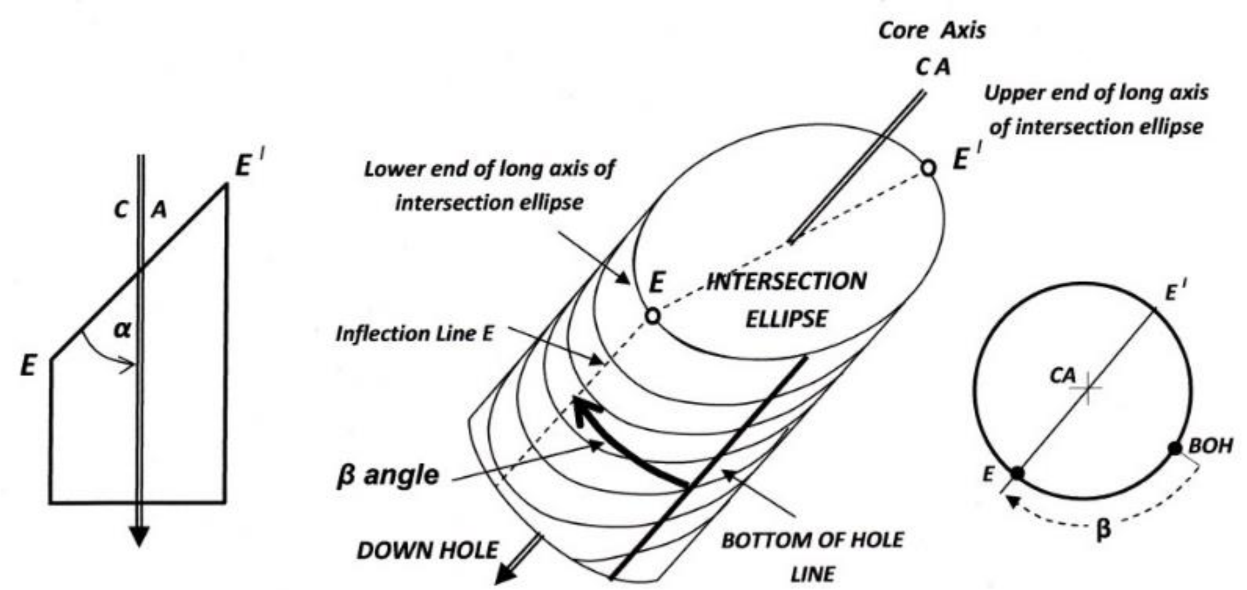
4.6 Spacing of discontinuity

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 C

Laboratory Test Results

AECOM Canada Ltd.
Winnipeg Geotechnical Laboratory
99 Commerce Drive
Winnipeg, Manitoba
R3P 0Y7
Phone: 204 477 5381 Fax: 204 284 2040

Project Name:	NEWPCC Biosolids Early Works	Supplier:	AECOM
Project Number:	60705950	Specification:	N/A
Client:	WSTP	Field Technician:	LBoughton
Sample Location:	Winnipeg, Manitoba	Sample Date:	October 27, 2023
Sample Depth:	Varies	Lab Technician:	LBoughton
Sample Number:	Varies	Date Tested:	November 1, 2023

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 (%)**
TH23-02	G1	0.15 - 0.30 m	**41.9%**				
	G2	0.61 - 0.76 m	**32.1%**				
	G3	1.37 - 1.52 m	**31.1%**				
	G4	2.13 - 2.29 m	**23.6%**				
	G5	2.90 - 3.05 m	**25.2%**				
	G6	3.66 - 3.81 m	**52.5%**				
	G7	4.42 - 4.57 m	**55.4%**				
	G8	5.94 - 6.10 m	**61.7%**				
	G9	7.47 - 7.62 m	**55.2%**				
	G10	8.99 - 9.14 m	**47.2%**				
	G11	10.52 - 10.67 m	**45.1%**				

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Project Name:	NEWPCC Biosolids Early Works	Supplier:	AECOM
Project Number:	60705950	Specification:	N/A
Client:	WSTP	Field Technician:	ABonifacio
Sample Location:	Winnipeg, Manitoba	Sample Date:	December 15, 2023
Sample Depth:	Varies	Lab Technician:	LBoughton
Sample Number:	Varies	Date Tested:	December 17, 2023

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 (%)**
TH23-08	G1	0.15 - 0.30 m	**55.5%**				
	G2	0.61 - 0.76 m	**42.4%**				
	G3	1.37 - 1.52 m	**35.7%**				
	T4	1.52 - 1.98 m	**35.6%**				
	G5	2.13 - 2.29 m	**24.6%**				
	G6	2.90 - 3.05 m	**48.3%**				
	G7	4.42 - 4.57 m	**54.2%**				
	T8	4.57 - 5.03 m	**53.4%**				
	G9	5.94 - 6.10 m	**55.2%**				
	G10	7.47 - 7.62 m	**50.2%**				
	T11	7.62 - 8.08 m	**46.2%**				
	G12	8.99 - 9.14 m	**43.8%**				
	G13	10.52 - 10.67 m	**46.7%**				
	T14	10.67 - 11.13 m	**38.2%**				
	G15	12.04 - 12.19 m	**52.3%**				
	G16	13.56 - 13.72 m	**53.4%**				
	G17	16.61 - 16.76 m	**54.0%**				
	S18	16.76 - 16.92 m	**34.2%**				

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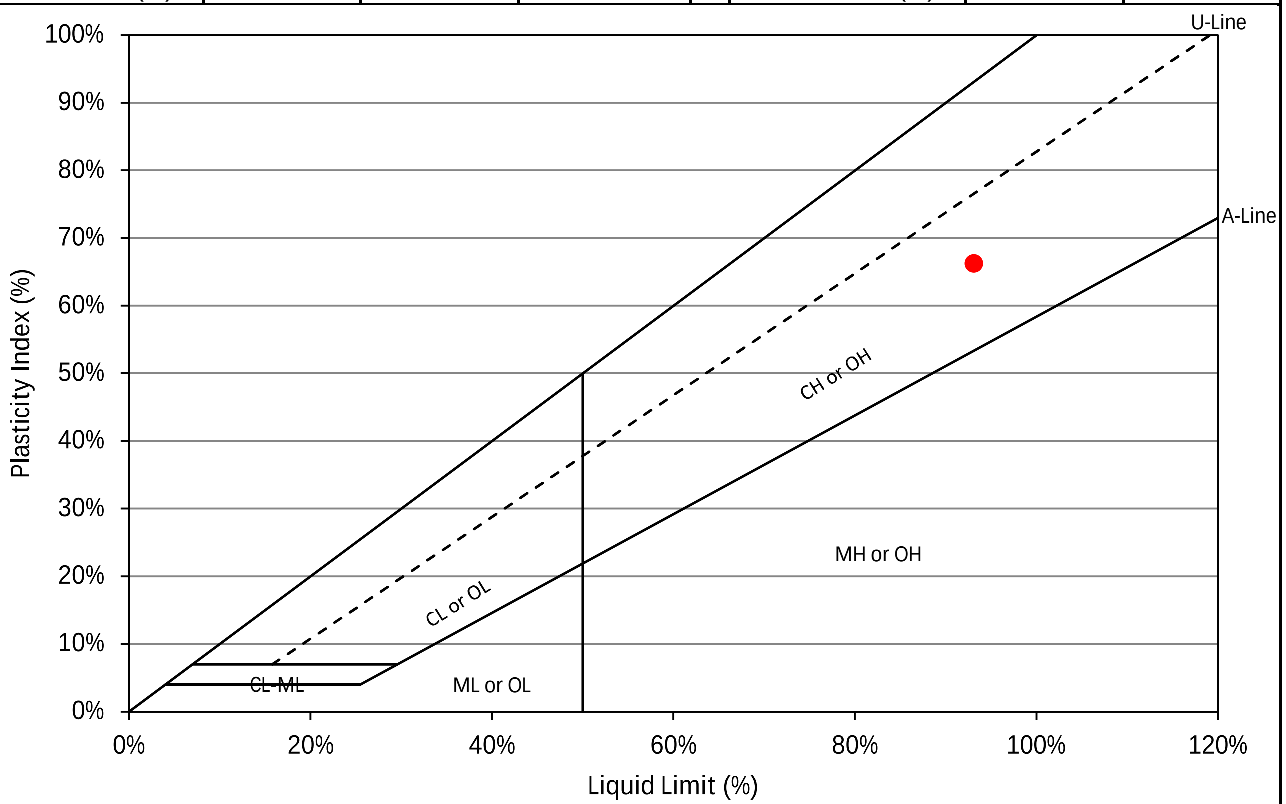
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	LBoughton
Sample Location:	TH23-02	Sample Date:	October 27, 2023
Sample Depth:	0.61 - 0.76 m	Lab Technician:	JWiens
Sample Number:	G2	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	33	20	18
Wet Sample (g)	10.5	7.7	8.6
Dry Sample (g)	5.5	4.0	4.4
Water Content (%)	91.0%	94.4%	95.9%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.0	4.7
Dry Sample (g)	4.8	3.7
Water Content (%)	26.4%	27.2%



Liquid Limit:	**93**	**Plastic Limit:**	**27**	**Plasticity Index:**	**66**

Reviewed by: Lee Boughton
Laboratory Manager

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

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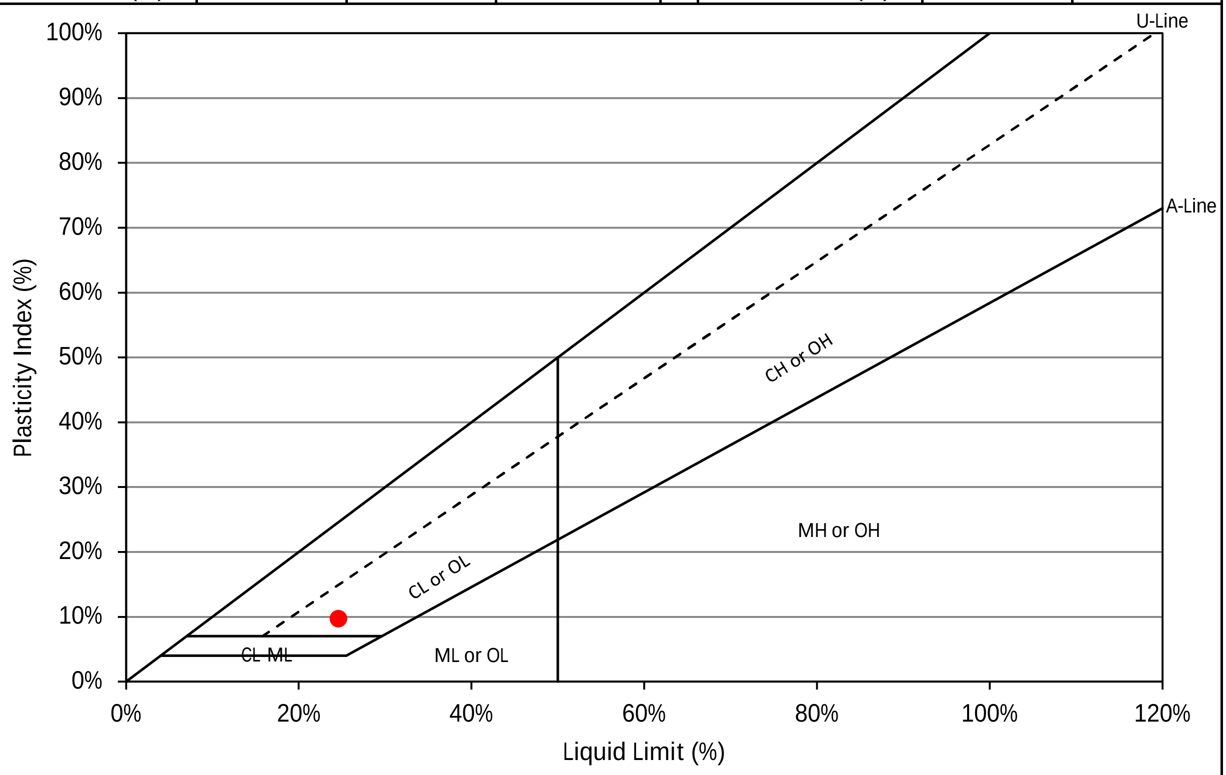
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	LBoughton
Sample Location:	TH23-02	Sample Date:	October 27, 2023
Sample Depth:	2.13 - 2.29 m	Lab Technician:	JWiens
Sample Number:	G4	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	34	24	17
Wet Sample (g)	9.4	10.7	9.6
Dry Sample (g)	7.6	8.6	7.7
Water Content (%)	23.8%	24.9%	25.3%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.2	9.0
Dry Sample (g)	5.4	7.9
Water Content (%)	14.7%	15.1%



Liquid Limit:	**25**	**Plastic Limit:**	**15**	**Plasticity Index:**	**10**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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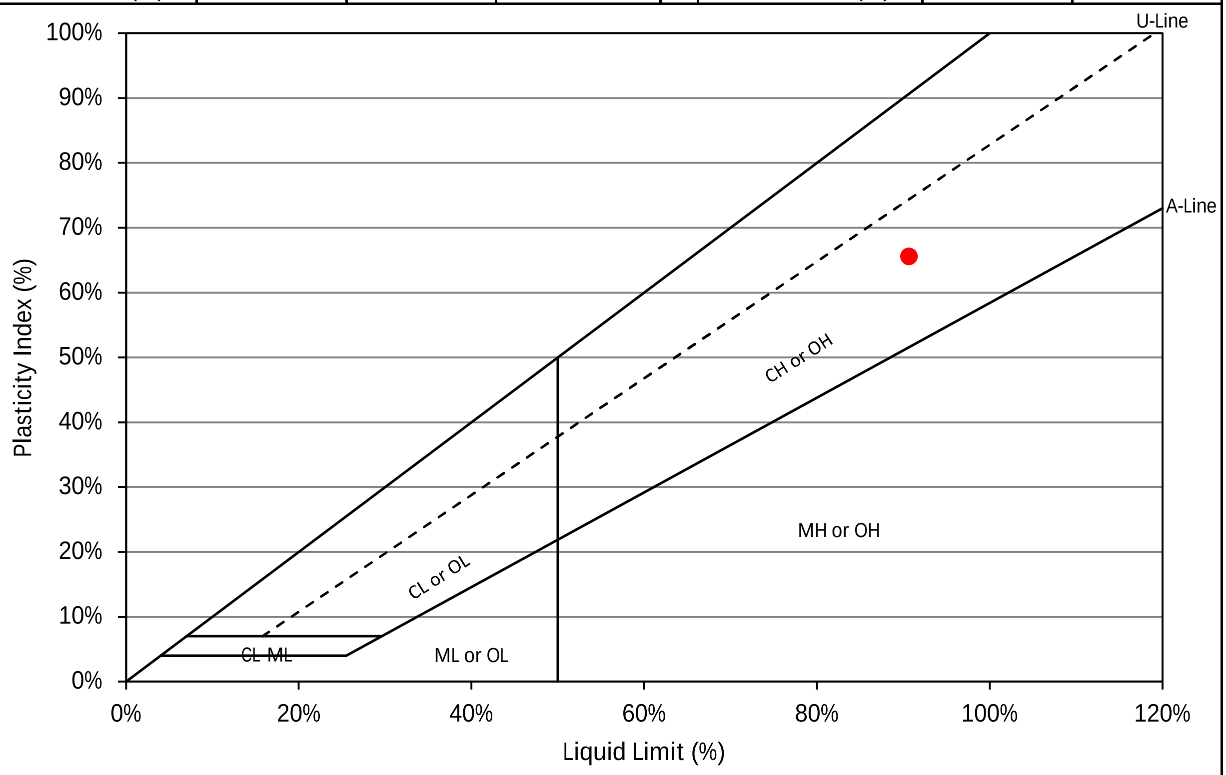
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	LBoughton
Sample Location:	TH23-02	Sample Date:	October 27, 2023
Sample Depth:	5.94 - 6.10 m	Lab Technician:	JWiens
Sample Number:	G8	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	35	23	17
Wet Sample (g)	7.8	9.2	9.4
Dry Sample (g)	4.1	4.8	4.9
Water Content (%)	89.4%	90.8%	92.2%

Plastic Limit		
Trial	1	2
Wet Sample (g)	5.9	5.9
Dry Sample (g)	4.7	4.7
Water Content (%)	25.1%	25.1%



Liquid Limit:	91	Plastic Limit:	25	Plasticity Index):	66

Reviewed by: Lee Boughton
Laboratory Manager

Approved by: German Leal, M.Eng., P.Eng.
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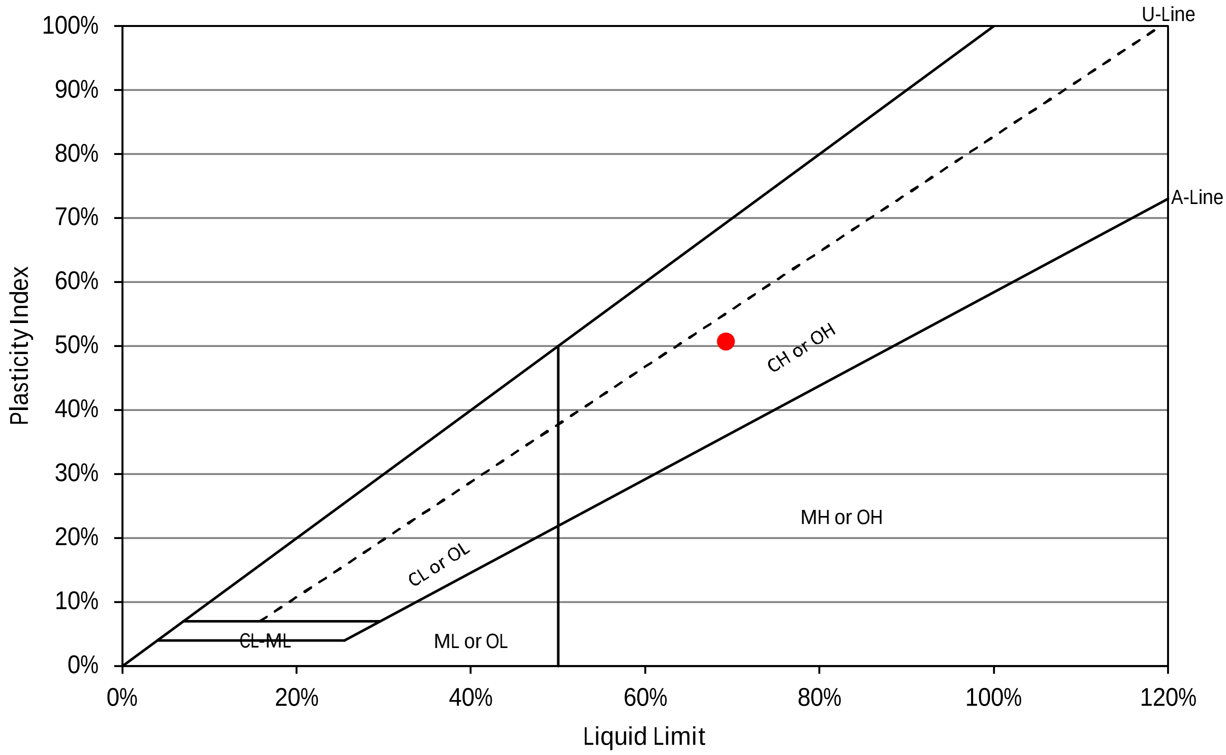
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	ABonifacio
Sample Location:	TH23-08	Sample Date:	December 15, 2023
Sample Depth:	1.52 - 1.98 m	Lab Technician:	JWiens
Sample Number:	T4	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	32	25	18
Wet Sample (g)	8.1	10.6	9.3
Dry Sample (g)	4.8	6.3	5.4
Water Content (%)	67.5%	68.4%	72.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.2	6.2
Dry Sample (g)	5.2	5.3
Water Content (%)	18.5%	18.4%



Liquid Limit:	**69**	**Plastic Limit:**	**18**	**Plasticity Index:**	**51**

Reviewed by: Lee Boughton
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Approved by: German Leal, M.Eng., P.Eng.
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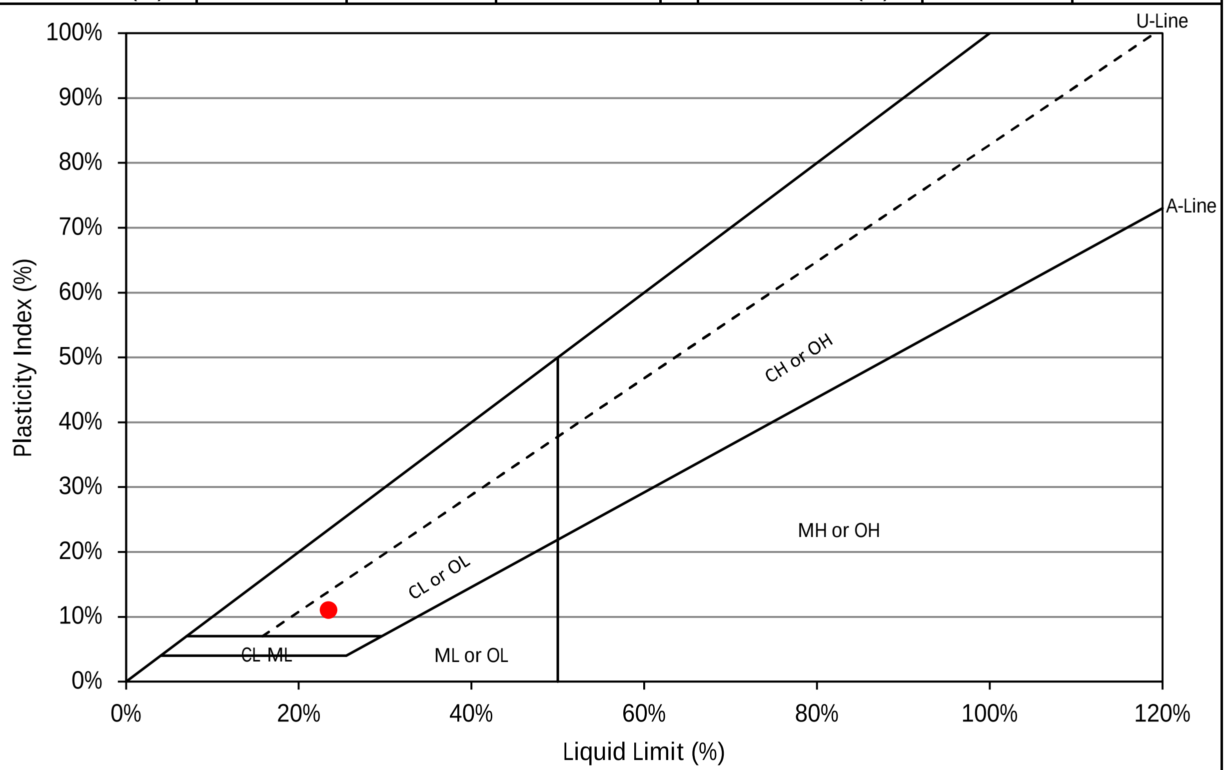
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	ABonifacio
Sample Location:	TH23-08	Sample Date:	December 15, 2023
Sample Depth:	2.13 - 2.29 m	Lab Technician:	JWiens
Sample Number:	G5	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	29	24	19
Wet Sample (g)	15.4	11.5	9.7
Dry Sample (g)	12.6	9.3	7.8
Water Content (%)	22.9%	23.4%	24.7%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.1	6.2
Dry Sample (g)	5.4	5.5
Water Content (%)	12.3%	12.5%



Liquid Limit (%):	**23**	**Plastic Limit (%):**	**12**	**Plasticity Index (%):**	**11**

Reviewed by: Lee Boughton
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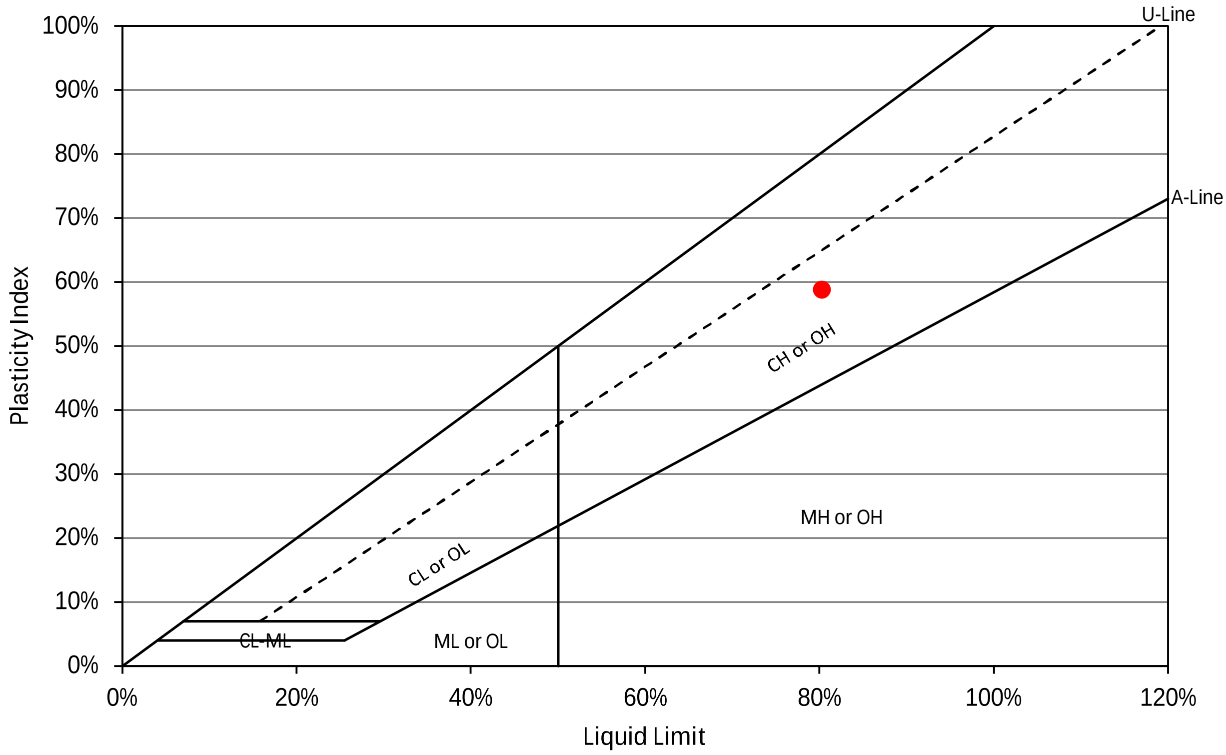
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	ABonifacio
Sample Location:	TH23-08	Sample Date:	December 15, 2023
Sample Depth:	7.62 - 8.08 m	Lab Technician:	JWiens
Sample Number:	T11	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	31	25	20
Wet Sample (g)	11.0	12.0	9.7
Dry Sample (g)	6.2	6.6	5.3
Water Content (%)	76.4%	81.9%	82.6%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.1	6.2
Dry Sample (g)	5.1	5.1
Water Content (%)	21.5%	21.2%



Liquid Limit (%):	**80**	**Plastic Limit (%):**	**21**	**Plasticity Index (%):**	**59**

Reviewed by: Lee Boughton
Laboratory Manager

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

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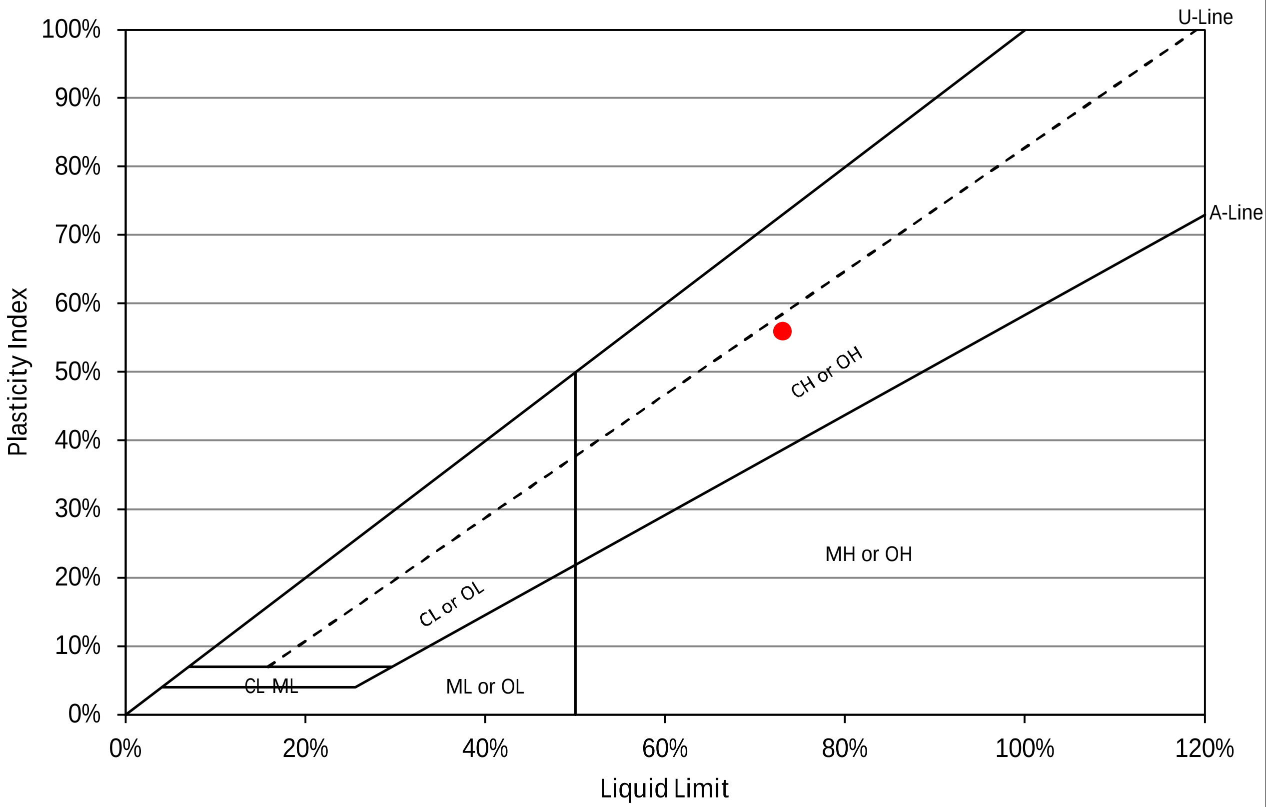
Project Name:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Supplier/Location:	Winnipeg, MB
Client:	WSTP	Field Technician:	ABonifacio
Sample Location:	TH23-08	Sample Date:	December 15, 2023
Sample Depth:	8.99 - 9.14 m	Lab Technician:	JWiens
Sample Number:	G12	Date Tested:	January 16, 2024

Atterberg Limits (ASTM D4318)

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

Liquid Limit			
Blows	33	23	17
Wet Sample (g)	8.5	8.9	13.0
Dry Sample (g)	5.0	5.1	7.4
Water Content (%)	71.4%	73.4%	75.8%

Plastic Limit		
Trial	1	2
Wet Sample (g)	6.0	6.1
Dry Sample (g)	5.2	5.2
Water Content (%)	16.9%	17.5%



Liquid Limit:	73	Plastic Limit:	17	Plasticity Index:	56

Reviewed by: Lee Boughton
Laboratory Manager

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

GRAIN SIZE DISTRIBUTION
(AASHTO T88)

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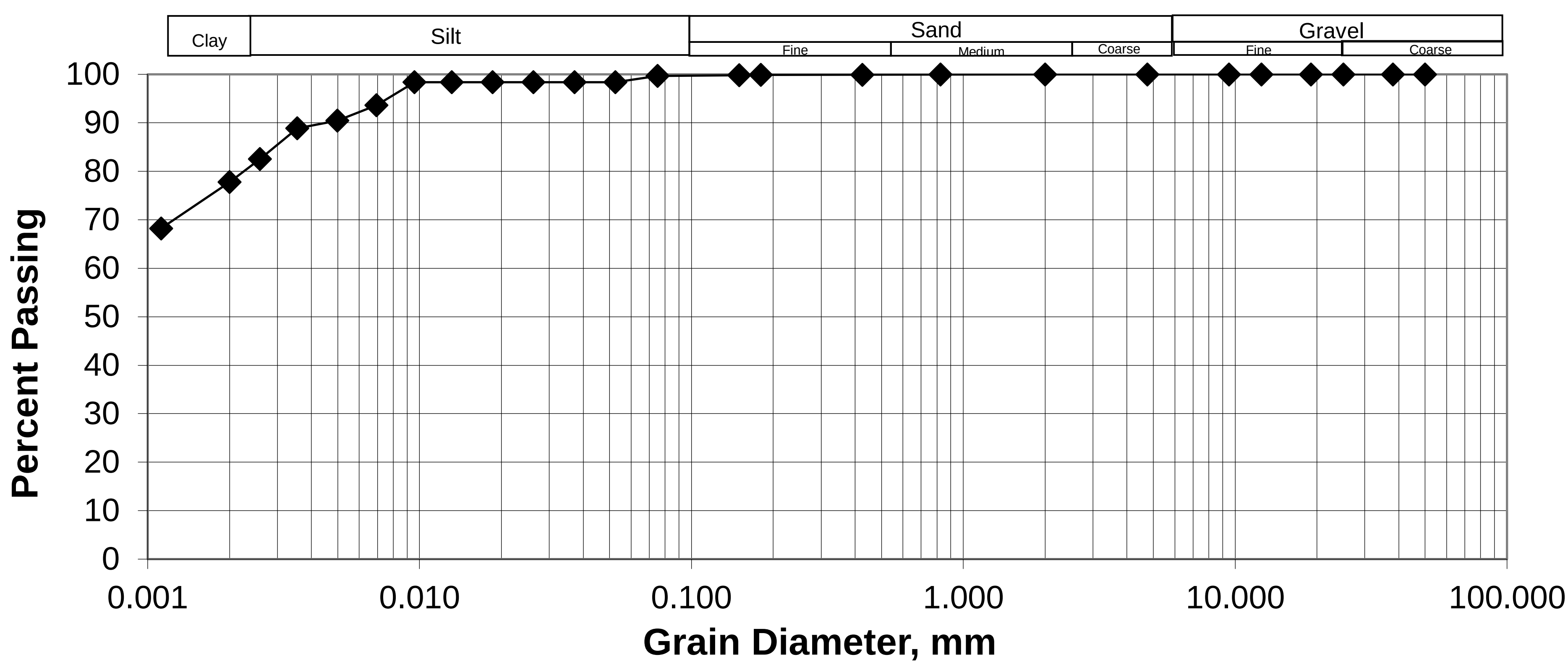
Job No.:	60705950	Hole No.:	TH23-02
Client:	WSTP	Sample No.:	G2
Project :	NEWPCC Biosolids Early Works	Depth:	0.61 - 0.76 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	100.0	0.0525	98.4
25.0	100.0	0.825	100.0	0.0371	98.4
19.0	100.0	0.425	99.9	0.0263	98.4
12.5	100.0	0.18	99.9	0.0186	98.4
9.5	100.0	0.15	99.8	0.0131	98.4
4.75	100.0	0.075	99.7	0.0096	98.4
				0.0069	93.6
				0.0050	90.5
				0.0036	88.9
				0.0026	82.5
				0.0020	77.8
				0.0011	68.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**21.9%**
Sand	**0.3%**	**Clay**	**77.8%**

Reviewed by: Lee Boughton
Laboratory Manager

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

GRAIN SIZE DISTRIBUTION
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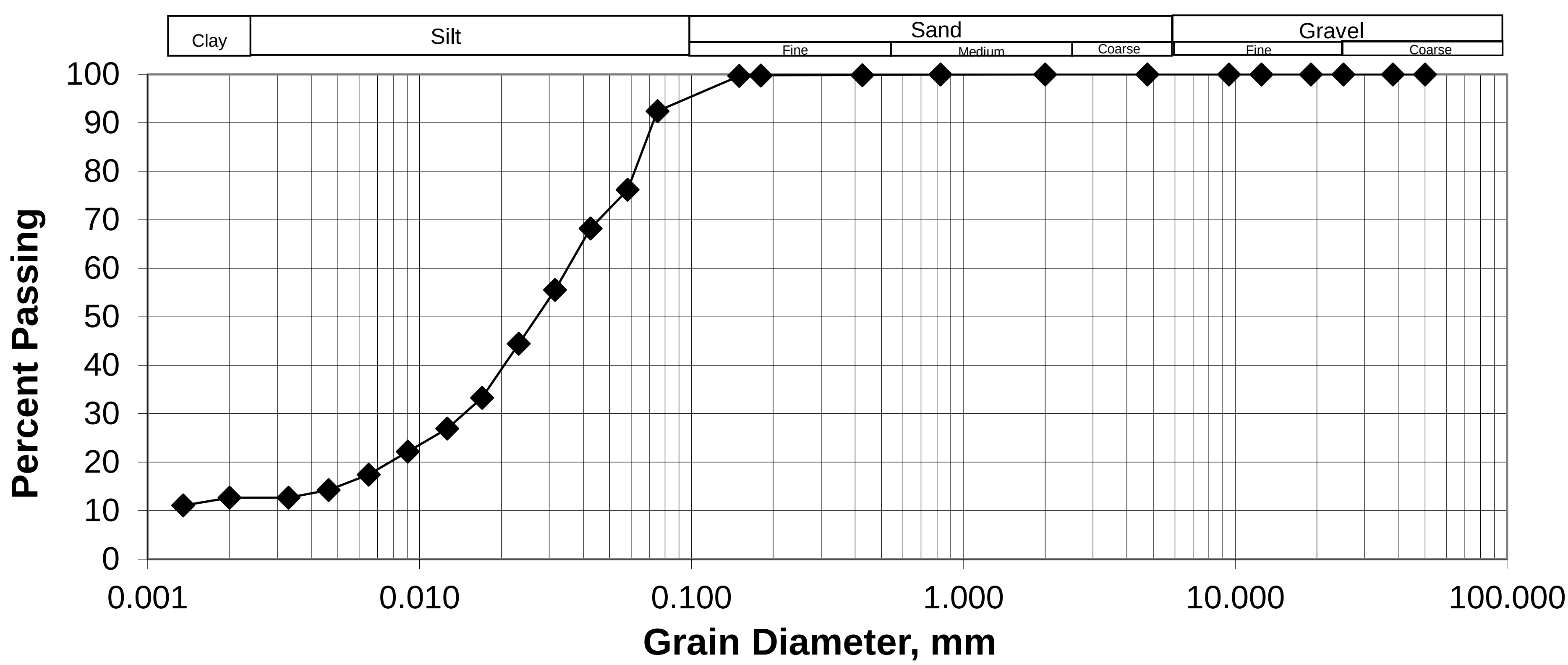
Job No.:	60705950	Hole No.:	TH23-02
Client:	WSTP	Sample No.:	G4
Project :	NEWPCC Biosolids Early Works	Depth:	2.13 - 2.29 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	92.4
38.0	100.0	2.00	100.0	0.0582	76.2
25.0	100.0	0.825	100.0	0.0425	68.2
19.0	100.0	0.425	99.9	0.0315	55.5
12.5	100.0	0.18	99.8	0.0232	44.4
9.5	100.0	0.15	99.7	0.0170	33.3
4.75	100.0	0.075	92.4	0.0126	26.9
				0.0091	22.2
				0.0065	17.4
				0.0046	14.2
				0.0033	12.6
				0.0020	12.6
				0.0013	11.1

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**79.8%**
Sand	**7.6%**	**Clay**	**12.6%**

Reviewed by: Lee Boughton
Laboratory Manager

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

GRAIN SIZE DISTRIBUTION
(AASHTO T88)

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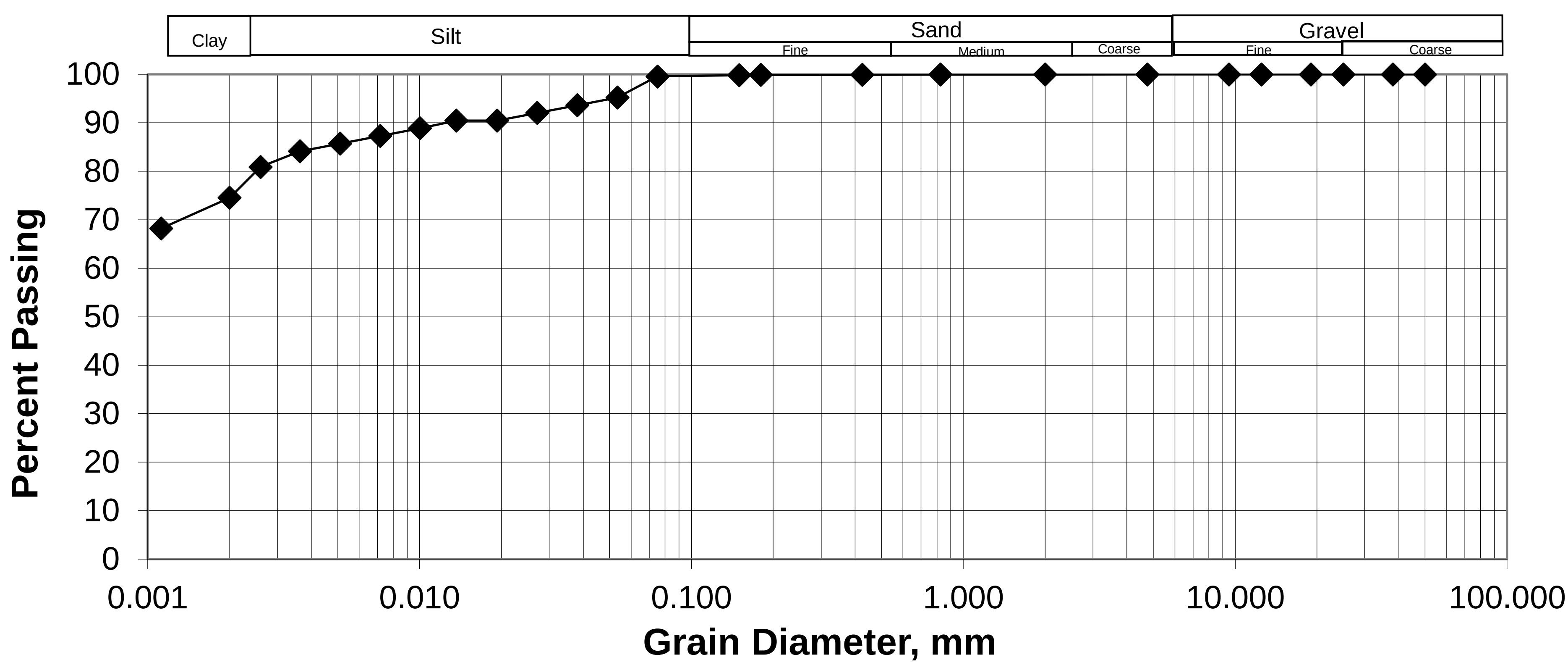
Job No.:	60705950	Hole No.:	TH23-02
Client:	WSTP	Sample No.:	G8
Project :	NEWPCC Biosolids Early Works	Depth:	5.94 - 6.10 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	99.6
38.0	100.0	2.00	100.0	0.0534	95.2
25.0	100.0	0.825	99.9	0.0380	93.6
19.0	100.0	0.425	99.9	0.0271	92.0
12.5	100.0	0.18	99.9	0.0193	90.4
9.5	100.0	0.15	99.8	0.0137	90.4
4.75	100.0	0.075	99.6	0.0100	88.9
				0.0072	87.3
				0.0051	85.7
				0.0036	84.1
				0.0026	80.9
				0.0020	74.6
				0.0011	68.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**25.0%**
Sand	**0.4%**	**Clay**	**74.6%**

Reviewed by: Lee Boughton
Laboratory Manager

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

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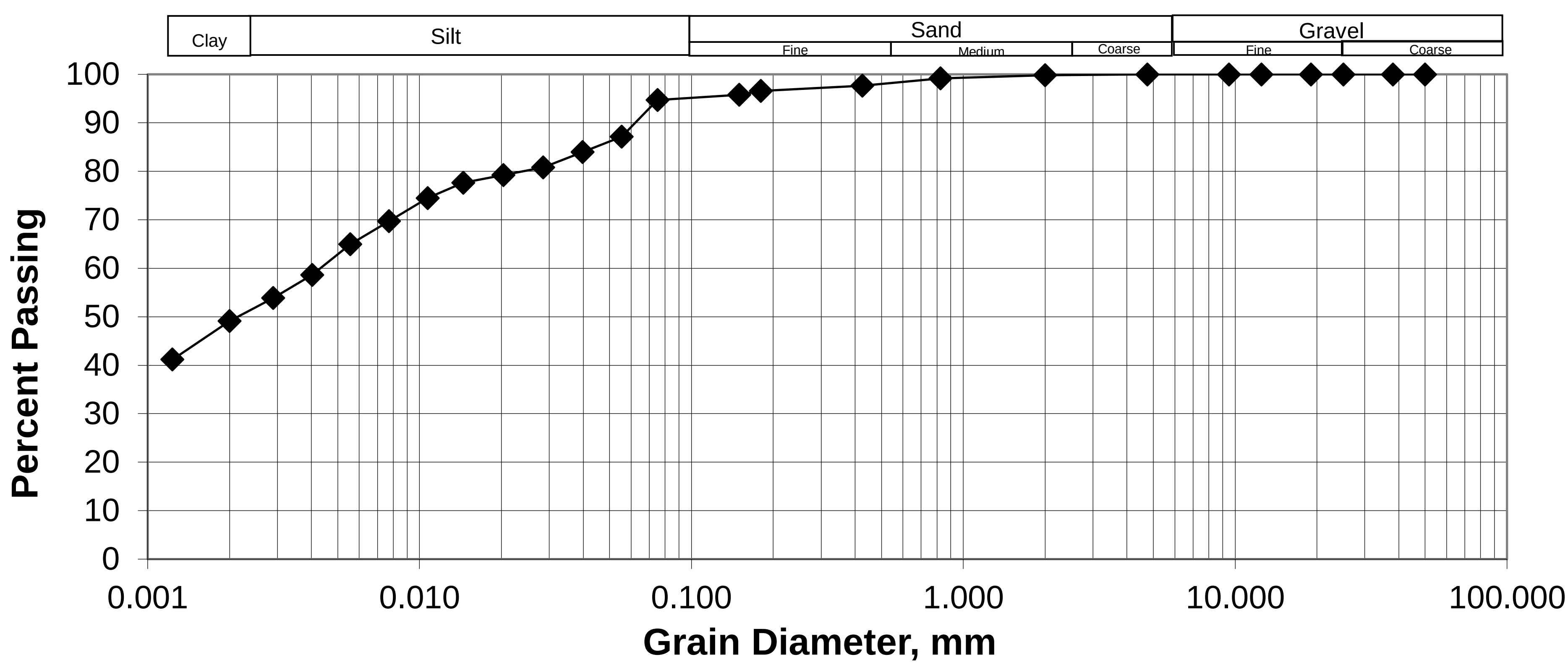
Job No.:	60705950	Hole No.:	TH23-08
Client:	WSTP	Sample No.:	T4
Project :	NEWPCC Biosolids Early Works	Depth:	1.52 - 1.98 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	94.7
38.0	100.0	2.00	99.8	0.0555	87.1
25.0	100.0	0.825	99.1	0.0398	84.0
19.0	100.0	0.425	97.7	0.0285	80.8
12.5	100.0	0.18	96.6	0.0203	79.2
9.5	100.0	0.15	95.8	0.0145	77.6
4.75	100.0	0.075	94.7	0.0107	74.5
				0.0077	69.7
				0.0056	64.9
				0.0040	58.6
				0.0029	53.8
				0.0020	49.1
				0.0012	41.2

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**45.6%**
Sand	**5.3%**	**Clay**	**49.1%**

Reviewed by: Lee Boughton
Laboratory Manager

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

GRAIN SIZE DISTRIBUTION
(AASHTO T88)

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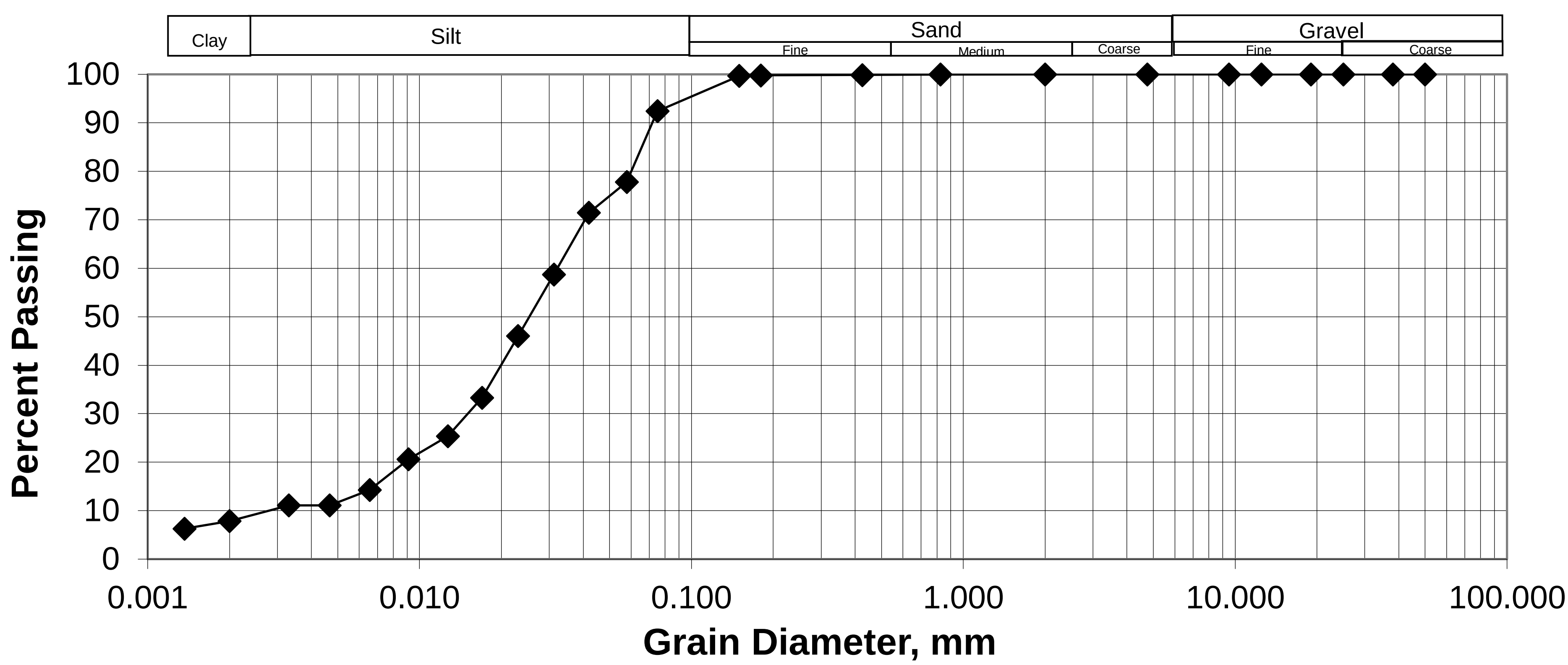
Job No.:	60705950	Hole No.:	TH23-08
Client:	WSTP	Sample No.:	G5
Project :	NEWPCC Biosolids Early Works	Depth:	2.13 - 2.29 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	92.4
38.0	100.0	2.00	100.0	0.0578	77.8
25.0	100.0	0.825	100.0	0.0420	71.4
19.0	100.0	0.425	99.9	0.0312	58.7
12.5	100.0	0.18	99.8	0.0230	46.0
9.5	100.0	0.15	99.7	0.0170	33.3
4.75	100.0	0.075	92.4	0.0127	25.3
				0.0091	20.6
				0.0066	14.2
				0.0047	11.1
				0.0033	11.1
				0.0020	7.9
				0.0014	6.3

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**84.5%**
Sand	**7.6%**	**Clay**	**7.9%**

Reviewed by: Lee Boughton
Laboratory Manager

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

GRAIN SIZE DISTRIBUTION
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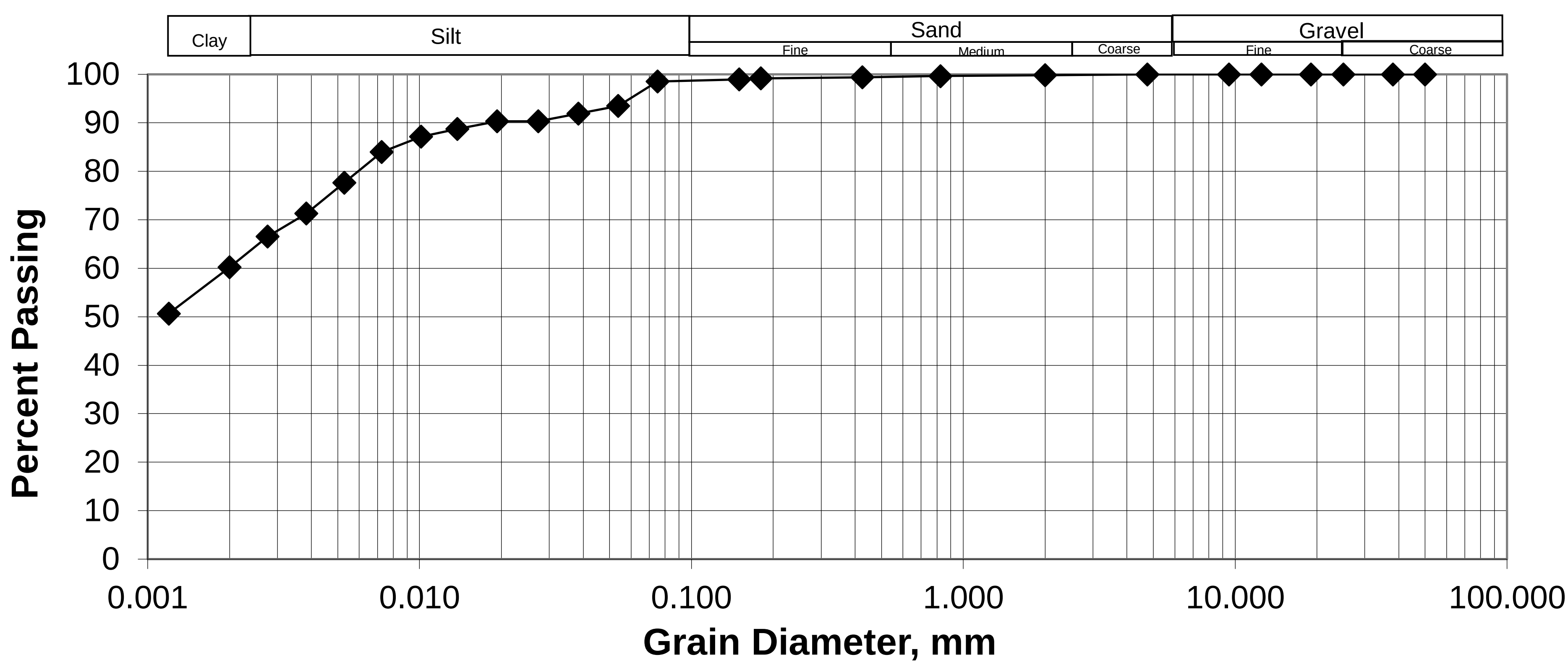
Job No.:	60705950	Hole No.:	TH23-08
Client:	WSTP	Sample No.:	T11
Project :	NEWPCC Biosolids Early Works	Depth:	7.62 - 8.08 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	98.5
38.0	100.0	2.00	99.8	0.0538	93.5
25.0	100.0	0.825	99.6	0.0383	91.9
19.0	100.0	0.425	99.4	0.0273	90.3
12.5	100.0	0.18	99.2	0.0193	90.3
9.5	100.0	0.15	98.9	0.0138	88.7
4.75	100.0	0.075	98.5	0.0101	87.1
				0.0073	83.9
				0.0053	77.6
				0.0038	71.3
				0.0028	66.5
				0.0020	60.2
				0.0012	50.7

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**0.0%**	**Silt**	**38.4%**
Sand	**1.5%**	**Clay**	**60.2%**

Reviewed by: Lee Boughton
Laboratory Manager

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

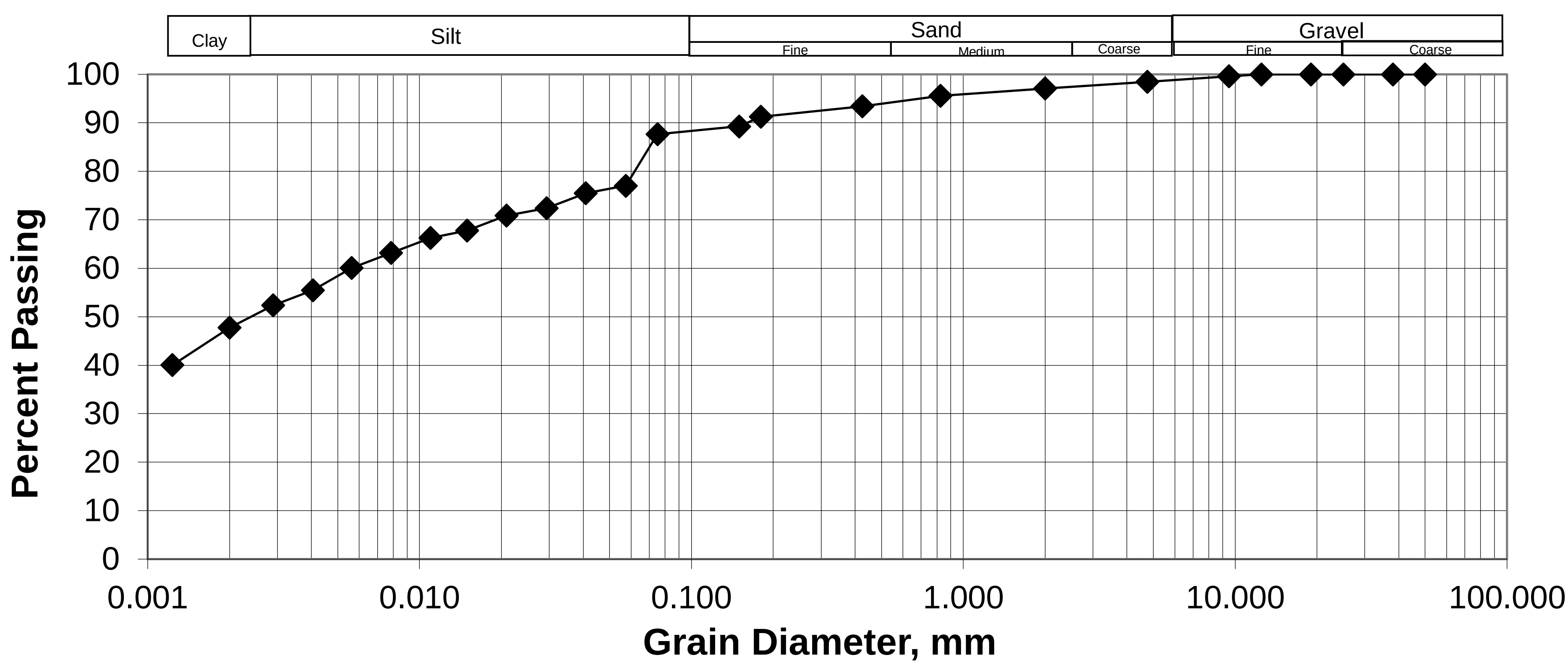
GRAIN SIZE DISTRIBUTION
(AASHTO T88)

AECOM
WINNIPEG GEOTECHNICAL LABORATORY
99 Commerce Dr., Winnipeg, MB R3P 0Y7 Canada
tel (204) 477-5381 **fax** (431) 800-1210

Job No.:	60705950	Hole No.:	TH23-08
Client:	WSTP	Sample No.:	G12
Project :	NEWPCC Biosolids Early Works	Depth:	8.99 - 9.14 m
Date Tested:	21-Nov-23	Date Sampled:	27-Oct-23
Tested By:	LBoughton	Sampled By:	LBoughton

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	98.5	0.0750	87.7
38.0	100.0	2.00	97.1	0.0575	77.0
25.0	100.0	0.825	95.6	0.0409	75.5
19.0	100.0	0.425	93.4	0.0293	72.4
12.5	100.0	0.18	91.3	0.0209	70.9
9.5	99.6	0.15	89.3	0.0149	67.8
4.75	98.5	0.075	87.7	0.0110	66.2
				0.0079	63.1
				0.0056	60.1
				0.0040	55.4
				0.0029	52.4
				0.0020	47.7
				0.0012	40.0

GRAIN SIZE DISTRIBUTION CURVE



Gravel	**1.5%**	**Silt**	**39.9%**
Sand	**10.8%**	**Clay**	**47.7%**

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:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Date Sampled:	December 15, 2023
Client:	WSTP	Sampled By:	LBoughton
Supplier/Location:	Winnipeg, MB	Date Received:	December 8, 2023
Sample Depth (m):	1.52 - 1.98 m	Submitted By:	LBoughton
Sample Location:	TH23-08	Date Tested:	January 8, 2024
Sample Number:	T4	Tested By:	LBoughton

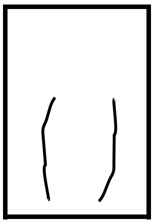
Unconfined Compressive Strength (ASTM D2166)

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

Soil Description:	CLAY/SILT - brown, firm, moist, trace sand, high plasticity, homogeneous

Average Diameter (cm):	7.16
Average Length (cm):	11.86
Length/Diameter Ratio:	1.66
Moisture content (%):	35.6
Bulk Density (g/cm³):	1.980
Bulk Unit Weight (kN/m³):	19.4
Bulk Unit Weight (pcf):	123.6
Dry Unit Weight (kN/m³):	14.32

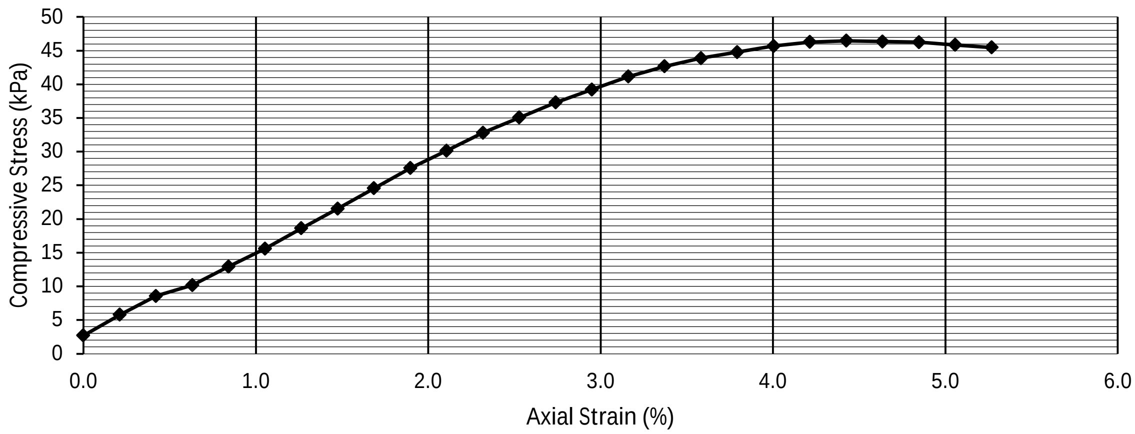
FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	-
Pocket Pen.	Undrained Shear Strength (kPa)	-

UCS	Unconfined compressive strength (kPa)	46.46	Undrained Shear Strength (kPa)	23.23
	Unconfined compressive strength (ksf)	0.970	Undrained Shear Strength (ksf)	0.485
	Avg. Rate of Strain to Failure (%/min):	1.26	Strain at Failure (%):	4.43

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:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Date Sampled:	December 15, 2023
Client:	WSTP	Sampled By:	LBoughton
Supplier/Location:	Winnipeg, MB	Date Received:	December 8, 2023
Sample Depth (m):	4.57 - 5.03 m	Submitted By:	LBoughton
Sample Location:	TH23-08	Date Tested:	January 8, 2024
Sample Number:	T8	Tested By:	LBoughton

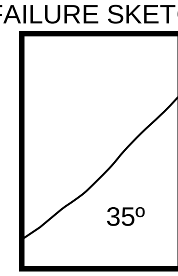
Unconfined Compressive Strength (ASTM D2166)

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

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

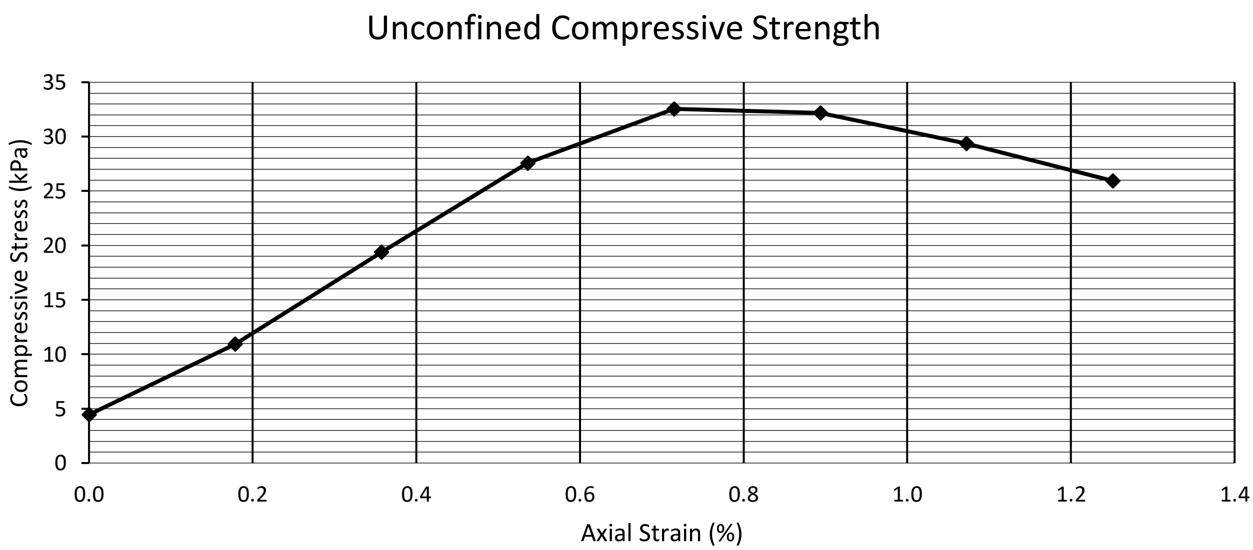
Average Diameter (cm):	7.16
Average Length (cm):	13.99
Length/Diameter Ratio:	1.95
Moisture content (%):	53.4
Bulk Density (g/cm³):	1.732
Bulk Unit Weight (kN/m³):	17.0
Bulk Unit Weight (pcf):	108.1
Dry Unit Weight (kN/m³):	11.07

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	53.9
Pocket Pen.	Undrained Shear Strength (kPa)	59.1

UCS	Unconfined compressive strength (kPa)	32.55	Undrained Shear Strength (kPa)	16.28
	Unconfined compressive strength (ksf)	0.680	Undrained Shear Strength (ksf)	0.340
	Avg. Rate of Strain to Failure (%/min):	1.07	Strain at Failure (%):	0.72



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:	NEWPCC Biosolids Early Works		
Project Number:	60705950	Date Sampled:	December 15, 2023
Client:	WSTP	Sampled By:	LBoughton
Supplier/Location:	Winnipeg, MB	Date Received:	December 8, 2023
Sample Depth (m):	7.62 - 8.08 m	Submitted By:	LBoughton
Sample Location:	TH23-08	Date Tested:	January 8, 2024
Sample Number:	T11	Tested By:	LBoughton

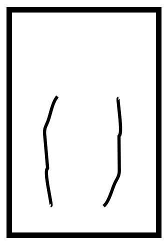
Unconfined Compressive Strength (ASTM D2166)

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

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

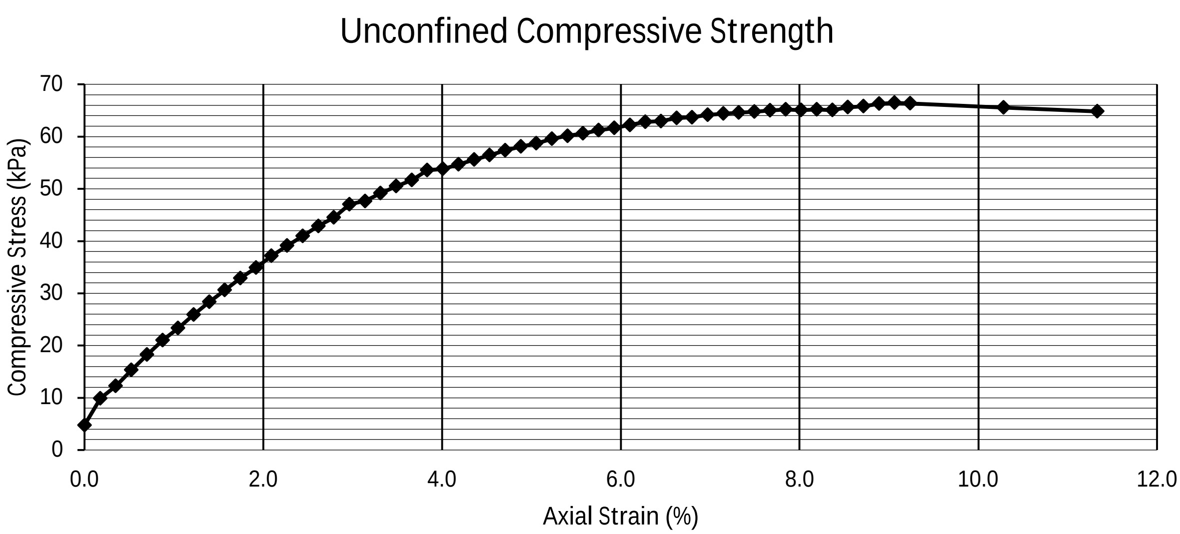
Average Diameter (cm):	7.17
Average Length (cm):	14.34
Length/Diameter Ratio:	2.00
Moisture content (%):	46.2
Bulk Density (g/cm³):	1.759
Bulk Unit Weight (kN/m³):	17.3
Bulk Unit Weight (pcf):	109.8
Dry Unit Weight (kN/m³):	11.80

FAILURE SKETCH



Torvane	Undrained Shear Strength (kPa)	-
Pocket Pen.	Undrained Shear Strength (kPa)	-

UCS	Unconfined compressive strength (kPa)	66.46	Undrained Shear Strength (kPa)	33.23
	Unconfined compressive strength (ksf)	1.388	Undrained Shear Strength (ksf)	0.694
	Avg. Rate of Strain to Failure (%/min):	1.05	Strain at Failure (%):	9.06



Comments:

Reviewed by: Lee Boughton
Laboratory Manager

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

Appendix D

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 (°)	49.952
Longitude (°)	-97.107

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

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.113	0.107	0.055	0.0216	0.00434	0.00126	0.0679	0.0544

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

NBC 2020 - 5%/50 years (0.001 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.0591	0.0565	0.028	0.0104	0.00193	0.000552	0.0339	0.027

The log-log interpolated 5%/50 year $S_a(4.0, X_E)$ value is : **0.0029**

NBC 2020 - 10%/50 years (0.0021 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.0334	0.0317	0.0149	0.00517	0.000881	0.000242	0.0184	0.0142

The log-log interpolated 10%/50 year $S_a(4.0, X_E)$ value is : **0.0014**

Download CSV

Go back to the seismic hazard calculator form

Date modified: 2021-04-06

German Leal, M.Eng., P.Eng.
Geotechnical Lead
E: german.leal@aecom.com

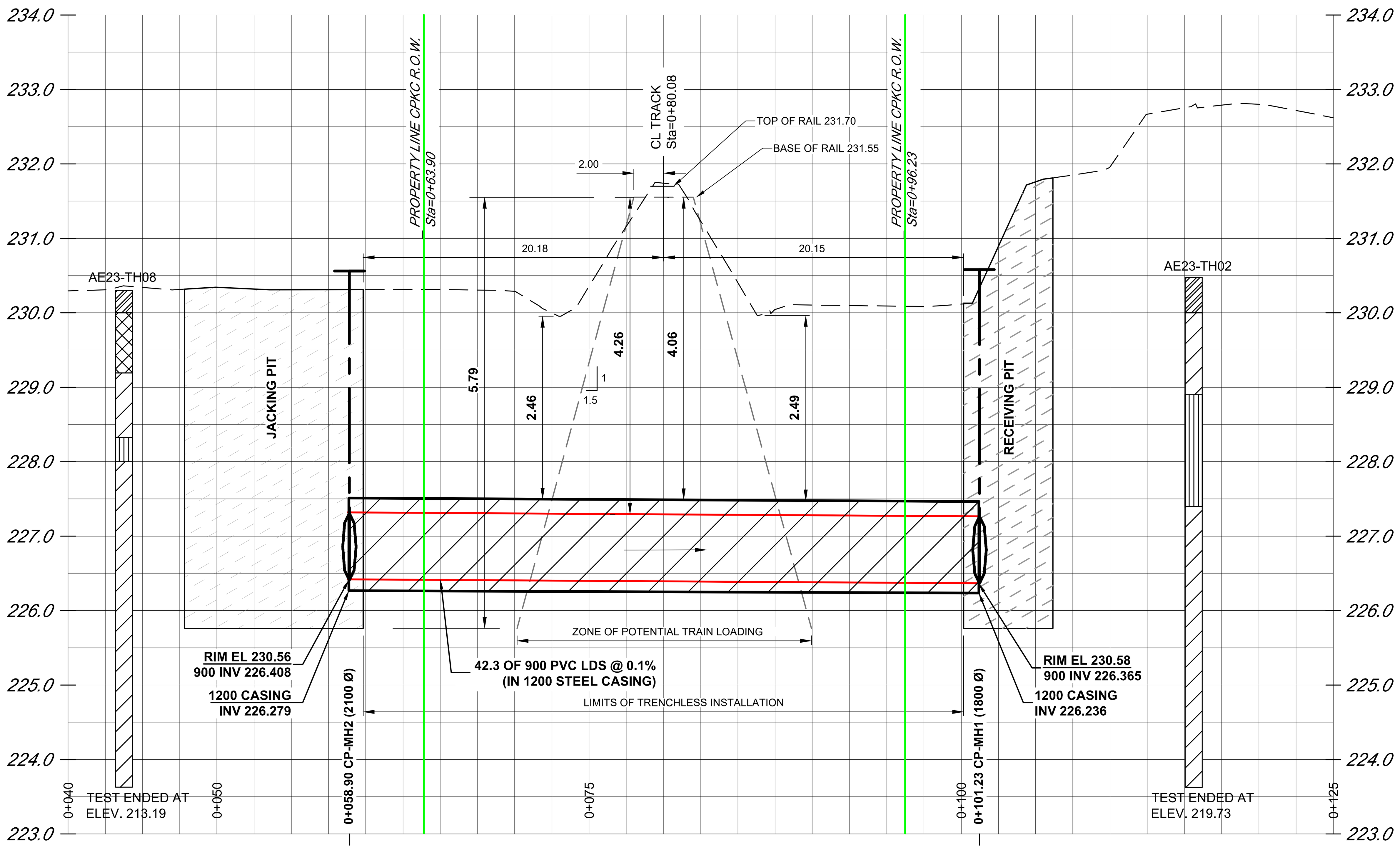
AECOM Canada Ltd.
99 Commerce Drive
Winnipeg, MB R3P 0Y7
Canada

T: 204.477.5381
F: 431.800.1210
www.aecom.com

Appendix II

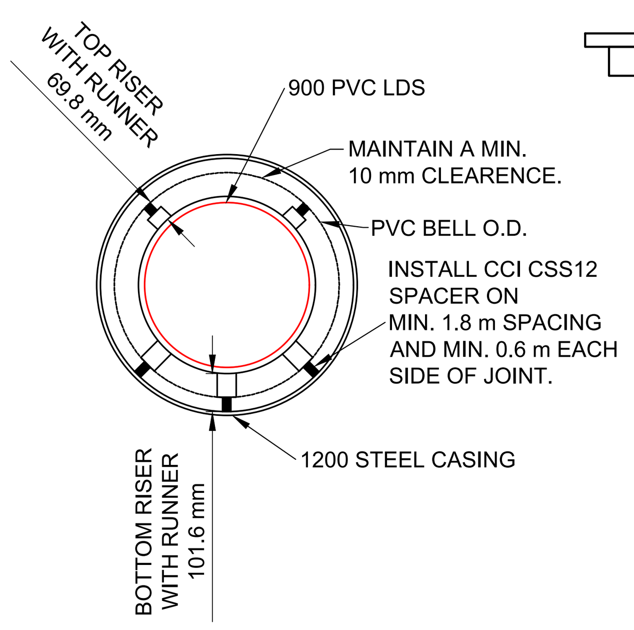
Design Drawings

REFER TO GEOTECH REPORT FOR GROUNDWATER ELEVATIONS.



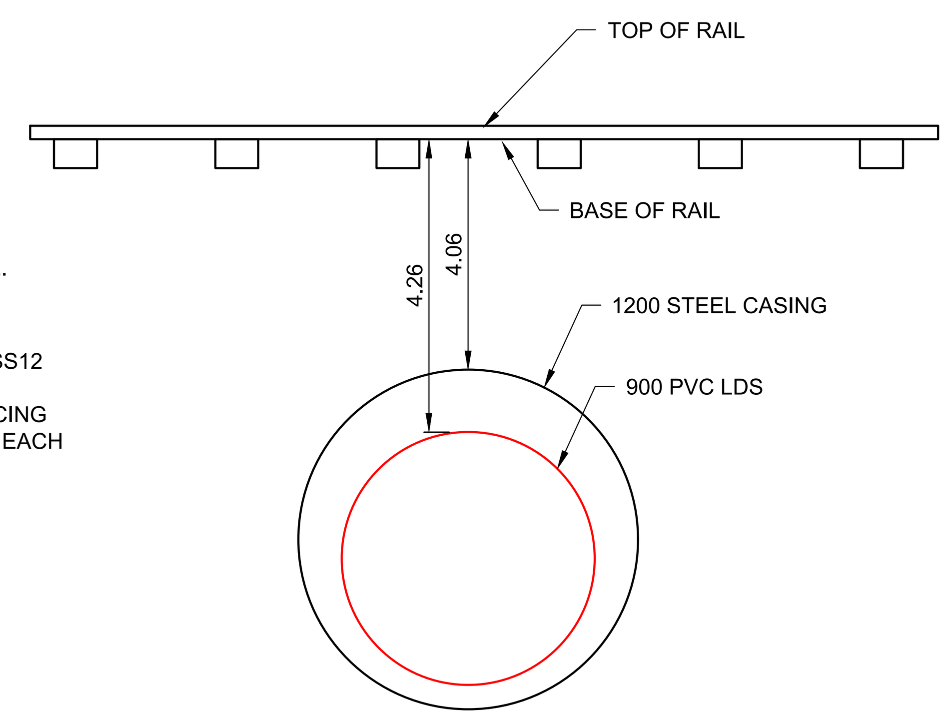
BOREHOLE LEGEND:

- TOPSOIL
- FILL
- LEAN CLAY
- FAT CLAY



CASING PIPE DETAIL

Scale NTS



SECTION ALONG TRACK

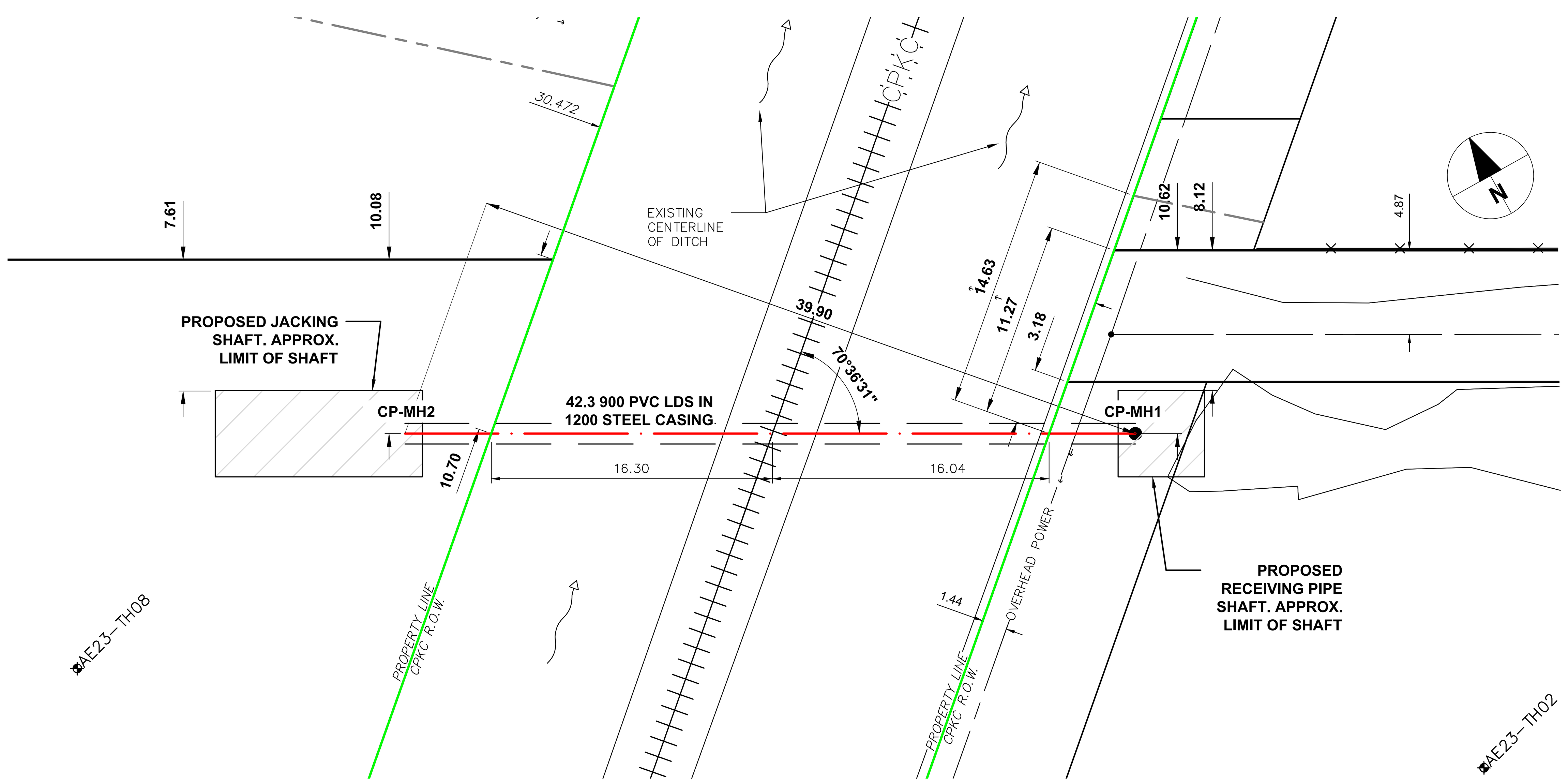
Scale NTS

RAIL CROSSING NOTES:

RAILWAY MILEAGE AND SUBDIVISION :	MILE 3.84, CP WINNIPEG BEACH SUBDIVISION	
MUNICIPAL DESCRIPTION OF ADJACENT PROPERTIES:	NORTH END WATER POLLUTION CONTROL CENTRE	
NAME OF PIPELINE OWNER:	CITY OF WINNIPEG, WATER AND WASTE DEPARTMENT.204-986-2626	
CONTENTS TO BE HANDLED:	LAND DRAINAGE	
MAXIMUM OPERATING PRESSURE:	N/A	
MAXIMUM SURGE AND TEST PRESSURE	N/A	
MAXIMUM OPERATING TEMPERATURE	- °C	
MINIMUM OPERATING TEMPERATURE	- °C	
PIPING MATERIALS	CARRIER	CASING
NOMINAL DIAMETER	914 mm	1184 mm
OUTSIDE DIAMETER	972 mm	1219 mm
MATERIAL	PVC	STEEL
SPECIFICATION AND GRADE	DR35	MIN. YIELD STRENGTH 241 MPa
WALL THICKNESS	27 mm	17.4 mm MIN.
TYPE OF JOINT	RUBBER GASKET BELL AND SPIGOT	WELDED
COATING	N/A	N/A
LOCATION OF SHUT OFF VALVES	N/A	
METHOD OF INSTALLATION	TRENCHLESS - AUGER BORE OR PIPE RAMMING WITH NO SOIL REMOVAL AHEAD OF PIPE	
VENTS:	NA / NUMBER: N/A	
	HEIGHT ABOVE GROUND: N/A	
SEALS:	BOTH ENDS: NO	
	TYPE: CONC. MANHOLE FOR FUTURE CONNECTION	
BURY:	BASE OF RAIL TO TOP OF PIPE CASING: 4.06 m	
	NOT BENEATH TRACKS: NA	
	ROADWAY DITCHES: 2.46 m (NW) 2.49 m (NE)	
	BASE OF RAIL TO BOTTOM OF JACKING/RECEIVING PITS: 5.79 m	
EXTENT OF CASING MEASURED PERPENDICULAR TO CENTERLINE OF TRACK	39.90 m	
ANGLE OF PIPE / TRACK CROSSING	70.36°	
DIRECTION OF FLOW	WEST TO EAST	
CENTERLINE OF TRACK TO FACE OF JACKING/RECEIVING PITS	20.18 m (N) 20.15 m (S)	
TYPE, SIZE, AND SPACING OF INSULATORS OR SUPPORTS	CCI CSS12 SPACERS 1.8 m O.C., 69.8 mm TOP RISER WITH RUNNER, 101.6 mm BOTTOM RISER WITH RUNNER	
CATHODIC PROTECTION	N/A	
GEOTECHNICAL BOREHOLES	REFER TO SOILS REPORT	
SOIL TYPE	REFER TO SOILS REPORT	
BASE OF RAIL TO GROUND WATER	REFER TO SOILS REPORT	

INSTALLATION AND MAINTENANCE SHALL BE IN ACCORDANCE WITH TC-E10 AND THE LATEST EDITION OF APPLICABLE CSA STANDARD Z662

GEOTECHNICAL ENGINEER EVALUATION HAS BEEN COMPLETED AND THE GEOTECHNICAL ENGINEER HAS DETERMINED THAT BASED ON THE SOIL CONDITIONS, THERE WILL BE NO ADVERSE EFFECTS TO CPKC OPERATION AND PROPERTY.



SITE PLAN

1:250

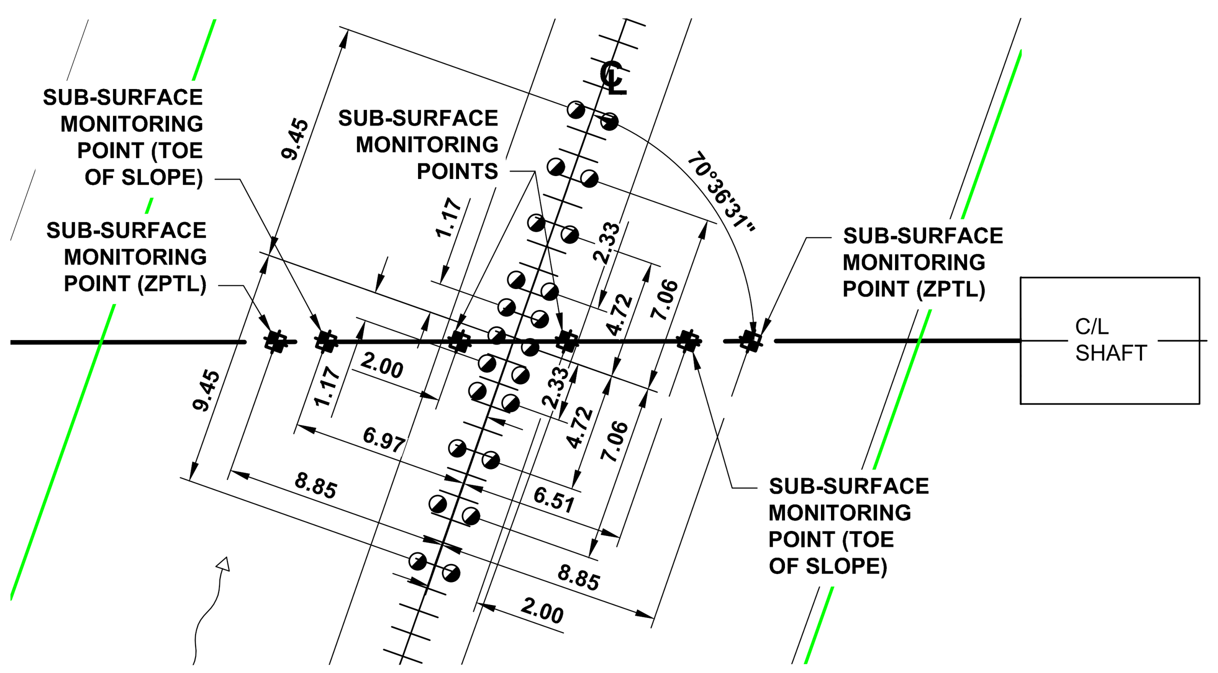
0 6.25 12.5 m

1:250

METRIC

WHOLE NUMBERS INDICATE MILLIMETRES
DECIMALIZED NUMBERS INDICATE METRES

SUB-SURFACE MONITORING POINTS (2) TO BE INSTALLED TO 1 m ABOVE CASING OBVERT IN ACCORDANCE WITH CP GEOTECHNICAL PROTOCOL FIGURE B



SUB-SURFACE/TRACK MONITORING PLAN

Scale 1:250

TRACK MONITORING

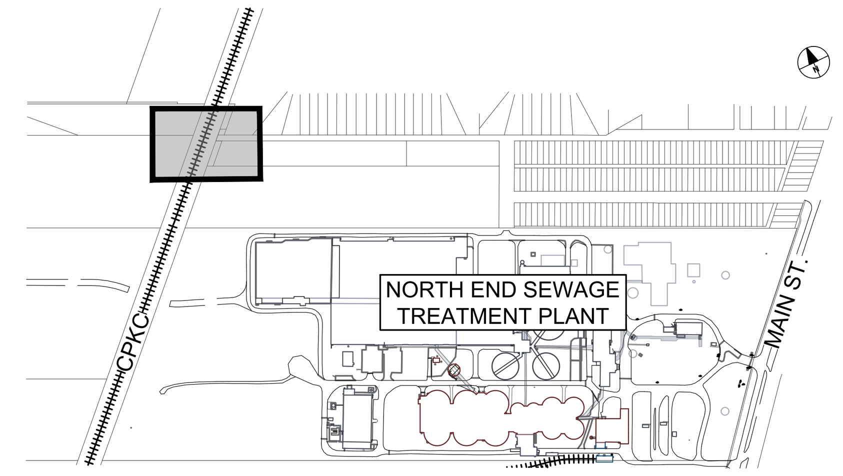
1. MONITOR RAIL TRACK IN ACCORDANCE WITH TRACK MONITORING PLAN, SEE SPECIFICATIONS.
2. REPORT MONITORING THRESHOLDS: CRITICAL / REVIEW THRESHOLD: ≥22 mm; ALERT / WARNING THRESHOLD: >11 mm
3. SETTLEMENT MONITORING OF THE RAIL TRACK SHALL COMMENCE A MINIMUM OF 2 DAYS PRIOR TO CONSTRUCTION/EXCAVATION IN ORDER TO ESTABLISH BASELINE COORDINATES AND ELEVATIONS.
4. MONITORING POINTS SHOULD BE SURVEYED AT LEAST TWICE PER DAY DURING THE DURATION OF CONSTRUCTION AND AT LEAST 3 DAYS AFTER THE COMPLETION OF CONSTRUCTION.
5. SURVEY RESULTS SHALL BE SUBMITTED DAILY TO GEOTECHNICAL ENGINEER FOR REVIEW.

900 PVC LDS PIPE ENDS TABLE

NORTHING	EASTING	CASING
5535452.27	635371.63	N. END 900 LDS
5535434.25	635409.93	S. END 900 LDS

CONTROL POINTS - NAD83 UTM ZONE 14 - JUNE 90 (LBIS)

POINT No.	NORTHING	EASTING	ELEVATION
BM 1	5534996.006	636000.318	231.073
BM 2	5534816.615	635804.484	230.670
BM 3	5535144.163	635145.506	232.293
BM 4	5535479.063	635036.233	231.014



KEY PLAN

TENDER NO. 116-2025

EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW	EXISTING	LEGEND - PLAN	NEW
150 WM	WATERMAIN	150 WM		CURB STOP			POLE	
	HYDRANT			REDUCER			ANCHOR	
	VALVE			COUPLING		230.850	GRADE	231.000
300 LDS	LAND DRAINAGE SEWER	300 LDS		HYDRO		230.50	CONTOUR ELEVATION	
250 WWS	WASTE WATER SEWER	250 WWS		MTS			SWALE	
	MANHOLE			GAS		PW	POTABLE WATER	PW
	CATCH BASIN			ASPHALT		FOA	FOUL AIR	FOA
	CURB INLET			CONCRETE		FSW	FLUSHING WATER	FSW
	SUBDRAIN			LIP CURB		SCP	SCUM PIPING	SCP
	FENCE			MODIFIED BARRIER CURB		HW	HYDRONIC WATER	HW
	SURVEY BAR			RAMP CURB		PD	PROCESS DRAIN	PD

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

ENGINEERS GEOSCIENTISTS MANITOBA
Certificate of Authorization
AECOM Canada ULC
No. 4671

NO.	REVISIONS	DATE	DESIGN	CHECK
B	ISSUED FOR REVIEW	2025-06-12	NJD	MG
A	ISSUED FOR CPKC APPROVAL	2025-02-21	NJD	MG

AECOM

DESIGNED BY: NJD | CHECKED BY: MG
DRAWN BY: JP | APPROVED BY: KWF
SCALE: 1:250 | RELEASED FOR CONSTRUCTION BY:
DATE: 2025-02-21 | DATE:
CONSULTANT NO.: C110

ENGINEER'S SEAL

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

NEWPCC - BIOSOLIDS EARLYWORKS
LAND DRAINAGE SYSTEM
CIVIL
LDS - CPKC CROSSING
WINNIPEG BEACH SUBDIVISION - MILE 3.84

CITY DRAWING NUMBER | SHEET 001 | REV. B | SIZE A1

A1 SIZE - 594mm x 841mm

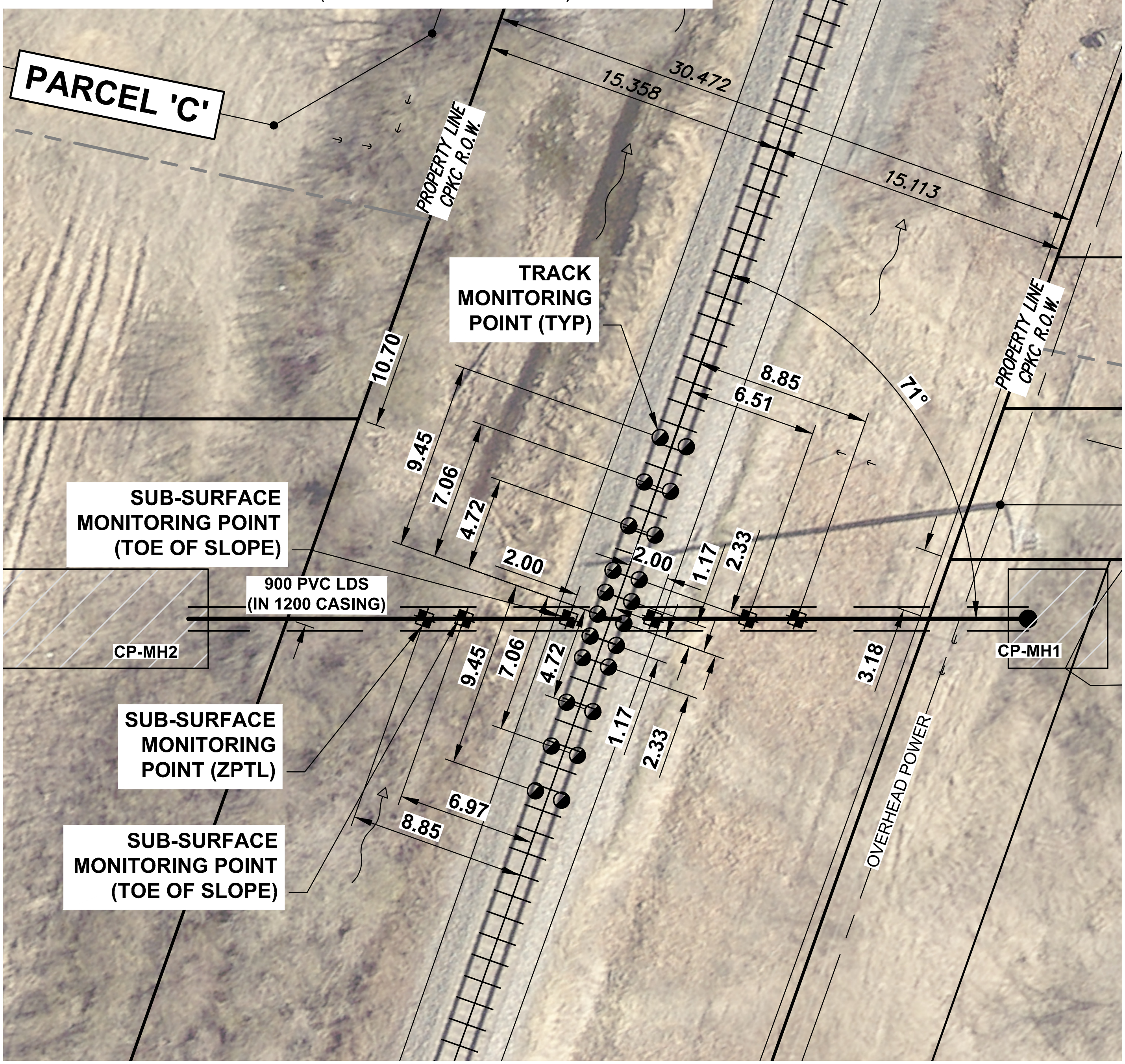
LAST SAVE: 2025/06/19 4:31 PM

PATH: C:\Users\moreirabezerral\DC\ACCDocs\AECOM\City of Winnipeg - NEWPCC Upgrades 2021\Project Files\900 Design Collaboration\60708950 Biosolids Earlyworks & Parcel C_EarlyWorks_LDS\C110_recover.dwg

LEGEND:

- SUB-SURFACE MONITORING POINTS
- SURFICIAL MONITORING POINT

NOTES:
THE SUBSURFACE MONITORING POINTS TO BE INSTALLED 1.0m ABOVE CASING PIPE OBVERT (BOTTOM ELEV. 228.50m)



Winnipeg

AECOM

Winnipeg Sewage Treatment Program
NEWPCC Upgrade : Biosolids Facilities
Early Works

LDS MONITORING POINT LOCATION PLAN
Figure: 1

Appendix III

CPR Protocol Geotechnical Protocol for Pipeline and Utility Installations within Railway Right-of-Way

DRAFT

GEOTECHNICAL PROTOCOL

FOR

PIPELINE AND UTILITY CROSSING(S) UNDER RAILWAY TRACKS

Engineering

Geotechnical and Utilities Department

Last Updated: May 15, 2024

4	May 15, 2024	DJW /JC / GD	7.1.1 added: retainer fee is Canada only 7.2 added: clarity on excavation pit placement 7.3, 7.3.1, 7.3.2 removed reference to Appendix A 9.2.2 added: clarity for 2m offset and end points of ZPTL 9.2.4 added: reference to Class 3/4/5 tracks for remote monitoring
3	March 12, 2024	DJW /JC / GD	Multiple sections highlighted throughout
2	May 16, 2022	DJW	Fig C – Pg 39
1	Feb 25, 2020	MR	Level 2 criteria – Pg 36
No	Date	By	Revision

CPKC Geotechnical Protocol for
Pipeline and Utility Crossing(s) under Railway Tracks

Last Updated: March 12, 2024

Engineering

Geotechnical & Utilities

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1.0 Limitations of the Document

The following protocol is independent of the requirements for assessing the structural components of the pipeline and pipeline crossing. The structural requirements for all pipeline crossings are included in SP-TS-2.39 - Pipeline and Cable Installations within Railway Right of Way. An agreement or permit from Canadian Pacific Kansas City Railway's Utilities Department is required before commencing with any work within the railway corridor. **Proposals for pipelines and utilities parallel to the track(s) are not covered under this protocol.**

In addition, this document does not cover review on any of the engineering design aspects of the proposed pipelines and utility crossings. Suitable engineering design is the sole responsibility of the applicant. Geotechnical approval of a proposed pipeline crossing by Canadian Pacific Kansas City (CPKC) in no way warrants the suitability of construction methods/techniques for anticipated ground conditions, nor does it warrant the suitability of existing ground and site conditions for the use proposed by the applicant of the crossing. CPKC does not take any responsibility for the suitability of the construction method or warrantee the ground and/or site conditions. CPKC geotechnical approval of a pipeline and utility installation application merely indicates that based on the provided and available information, the proposed construction and design addresses CPKC's needs at the time of review and approval. CPKC does not provide engineering recommendations, directions or minimum standards to the applicant or their contractor(s) for design and execution of their work within CPKC Right-of-Way (ROW).

Due to third party work on CPKC ROW, CPKC will not be liable for any damages or delays to the applicant and/or CPKC assets and operation because of its approval of an application. In addition, any damages incurred to CPKC due to third party pipeline and utility crossing(s) will be the responsibility of the applicant.

CPKC requires that the applicant provide adequate documentation as outlined in this protocol; clearly identify the responsible Professional Engineer of Record and the components of the project for which they are responsible.

2.0 General Terminology

Base of Rail (BOR): is the bottom surface of the rail and is frequently used as a local datum from which vertical measurements are referenced. If an external datum is utilized the elevation of the BOR will be identified.

Geotechnical Engineer of Record's onsite designate/representative: A geotechnical trained and competent person assigned by the Geotechnical Engineer of Record to act as site inspector who will be present onsite during the full duration of the construction and installation within railway operating corridor, unless, otherwise directed by CPKC Utilities Supervisor. The site inspector must have the required training, experience and understanding of the site conditions, proposed design, and construction methodology to make sound engineering judgement and decisions, and reports during the course of the work.

Service Provider(s): include professional engineering firm(s) or individual(s) representing relevant or applicable engineering disciplines, to be retained on behalf of CPKC for engineering related review and/or oversight of fieldwork and track settlement monitoring results, for which the compensation will be paid by the applicant.

Zone of Potential Track Loading (ZPTL): is considered as the area under the track and within a 1V to 1.5H soil zone extending down from a point at the level of the BOR and 2 m (6.6 ft.) from The centerline of track as shown in Figure 3.

FRA: Federal Railroad Administration.

TC: Transport Canada.

3.0 Introduction

The purpose of this document is to ensure efficient application process and ensure safety and uninterrupted operation of Canadian Pacific Kansas City (CPKC) Railway's operations during the execution of proposed third party pipeline and utility crossing(s) within CPKC ROW. This document is intended to guide the applicant of the minimum application requirements, review and approval process for proposed pipeline and utility crossing(s) as completed by CPKC Utilities and Geotechnical groups. The goal of this protocol is to:

3.1 Provide safe track(s) conditions during and after the installation of proposed pipeline and utility crossing(s);

3.2 Establish requirements and procedures to be followed by the applicant(s) to minimize difficulties and risks to CPKC's operations and its assets during the installation and operation of pipeline and utility crossing(s) under CPKC's tracks and within its ROW;

3.3 Specify minimum criteria to be met for CPKC's review;

3.4 Ensure adequate subsurface information including geotechnical and groundwater information is available and an assessment by CPKC's geotechnical group or a CPKC approved service provider has been completed prior to providing approval; and

3.5 Allow timely processing of application for pipeline and utility crossing(s) approvals.

4.0 Emergencies

In the event of any occurrence due to construction/contractor activities that does or could pose a hazard, immediately contact CPKC Police at **1-800-716-9132**.

5.0 Winter Work Restriction within CPKC ROW

No construction and installation of pipeline and utility crossing(s) that fall under the Geotechnical Protocol will take place between December 15th and March 31st. This restriction is particularly critical to areas where frost penetrates the ground and may make it difficult to observe surface settlement and loss of soil from underneath the track substructure due to misperception of a levelled frozen surface. Such conditions pose a risk to the stability of CPKC's track and its substructure during thawing season and are not acceptable.

In areas where the applicant does not consider frost as a potential risk, the applicant is required to assure and demonstrate to CPKC as to why winter work restriction is not applicable to their proposed work. Exceptions to winter work restriction will be evaluated on case by case basis.

6.0 Application Process Identification

To identify the applicable process, complete appropriate level of assessment and allow timely processing of a pipeline and utility crossing(s) proposal, the requirement criteria have been divided into three levels as identified in Table 1, i.e. Minimum, Intermediate and Detailed. These processes are categorized based on the size, cover, location and proximity of pipeline from tracks and other structures, and construction methodology of the proposed pipeline and utility crossing(s).

Applicant is expected to consult Table 1 to identify the level of effort and detail of submission required to meet CPKC review requirements for review. Details of each process are discussed in the following sections.

Table 1 – Process Identification

		Process Levels		
		1. Minimum[1]	**2. Intermediate**	**3. Detailed**
Dimension Criteria	Outside diameter of pipe	Less than 300 mm (12 in.)	300 mm (12 inches) to 1500 mm (59 in.)	Greater than 1500 mm (59 in.)
	Cover between BOR and top of pipe	Greater than 1.5 m (5 ft.) or three pipe diameters whichever is greater.	Greater than 1.5 m (5 ft.) or two (2) pipe diameters whichever is greater.	Less than 1.5 m (5 ft.) or two (2) pipe diameters.
	Adjacent structures including switches and signals	Greater than 10 m (32.8 ft.) from centerline	Within 2.5 times, cover between BOR and top of pipe.	
	Depth of pipes outside ZPTL	Refer to SP-TS 2.39 All pipes will be at least 0.91 m (3 ft.) below ground (below sub-ballast layer) where pipes are not below the ZPTL.	Less than 0.91 m (3 ft.) burial within ZPTL.	
Excavation Criteria	Excavation close to CPKC track(s)	Jacking/access pits shall be more than 10 m (32.8 ft.) from the closest track centerline and shall not encroach on the ZPTL.	Excavations or jacking/access pits within 10 m (32.8 ft.) of the closest track centerline.	
	Crossing angle	Less than 45 degrees off perpendicular to the track.	More than 45 degrees off perpendicular to the track.	

	Process Levels		
	1. Minimum[1]	**2. Intermediate**	**3. Detailed**
Construction Method	Trenchless method[2]		All methods considered.
	Pipe bursting will only be considered where the predicted heave is less than 10% of the movement that would result in a change of the FRA or TC track class.		
Approval Process	Utility group to approve with no geotechnical submission.	Full review of design, geotechnical and construction method Applicant to pay for the review cost of CPKC approved service provider.	

[1] Move to next class if one or more criteria are not met.

[2] Trenchless methods include Auger Boring (AB), Pipe Jacking, Pipe Ramming (PR), Horizontal Directional Drilling (HDD) except high pressure fluid jetting method, Microtunnelling (MT) but exclude any type of mining techniques where any stand up time is required before the tunnel support is placed.

7.0 Minimum Information Requirements

7.1 All proposals for pipeline and utility crossing(s) approvals will be under the signature and seal of a locally registered professional Geotechnical Engineer referred to as Geotechnical Engineer of Record (GER). The objective is to ensure that a registered professional from applicant's design firm or organization is given the opportunity and responsibility to assess the site and subsurface conditions and demonstrates due diligence to assure CPKC that the proposal is appropriate for such conditions. This, however, depending on the complexity of design and proposal, may be in addition to the requirements of meeting industry standards or current regulatory requirements for structural integrity of the pipeline/utility. Such design will also require signature and seal by a professional geotechnical and/or structural engineer.

7.1.1 All applications to which the CPKC Geotechnical Protocol applies must include a separate retainer fee to cover costs incurred to the railway due to the project's activities, (such as but not limited to) resurfacing work, survey to obtain as-built drawings, site cleanup, and removal of settlement monitoring equipment. Retainer fee (or a portion thereof) is refundable if final stamped geotechnical construction summary report and stamped as-built drawing(s) are provided within six (6) months of completion of construction and post-construction monitoring. Retainer fee only applies to applications in Canada.

7.2 The application package must include a construction plan that specifies the terms and conditions for the execution of the proposed work, including assignment of responsibility. The applicant of the crossing(s) is responsible to ensure that the work is executed in accordance with the terms of the agreement with CPKC. The drill path and installation of the jacking and receiving pits should be planned to have the least impact to railway operations. The jacking and receiving pits should be placed outside CPKC property and not be planned or excavated within the (ZPTL) - zone of potential track loading. The access pits can be closer to the tracks if the grades and soil conditions call for it and if it also reduces the chances of voids or track settlement, but will require review of the specific site. Any exceptions to the placement of the pits will require additional reviews at the applicant's expense

7.3 **Engineering Drawings:** All pipeline and utility crossing(s) application packages will be accompanied by following documents, at minimum, showing features on drawings in true scale.

7.3.1 Plan of the proposed pipe and utility crossing(s) under the track. This drawing will show the following features:

7.3.1.1 Location of the crossing(s), referencing identifiable landmarks including Mileage and Subdivision of the proposed crossing(s) as per CPKC Subdivision naming and Mileage convention. Applicant can obtain the Mileage and Subdivision information from CPKC Utilities group; The title of the plan will include the subdivision name and mileage of the location.

7.3.1.2 Pipe centerline, diameter, length, size, limits, thickness and material;

7.3.1.3 Location of any adjacent structures including but not limited to signals, switches, culverts, other existing underground/buried services including Fibre Optics Transmission Systems (FOTS) and relevant distances from the centerline of the track(s);

7.3.1.4 Location of the ditch line and any breaks in slope;

7.3.1.5 Location of drilled boreholes or test pits from geotechnical investigation;

7.3.1.6 Location of all tracks and distances from track centerline to the proposed work area location; and

7.3.1.7 Location of all access pits, size, depth and details of support of excavation, if applicable.

7.3.2 Profile of the track and proposed pipeline and utility crossing(s) along the centerline of the track. The profile will show the following features:

7.3.2.1 Location of the crossing(s), referencing identifiable landmarks including Mileage and Subdivision of the proposed crossing(s) as per CPKC Subdivision naming and Mileage convention. Applicant can obtain the Mileage and Subdivision information from CPKC Utilities Group;

7.3.2.2 Pipe centerline, diameter, length, size, limits, thickness and material;

7.3.2.3 Location of any adjacent structures including but not limited to signals, switches, culverts, other existing underground/buried services including Fibre Optics Transmission Systems (FOTS) and vertical distance from BOR;

7.3.2.4 Elevation of surface water in ditches, elevation of the ground water table in all boreholes and the date it was measured;

7.3.2.5 Test pit/borehole locations along with the stratigraphic profile as determined through the geotechnical investigation;

7.3.2.6 Depth of top of pipe to the BOR; and

7.3.2.7 Location of all jacking/access pits, size, depth and details of support of excavation, if applicable.

7.3.3 Section of the track along the centerline of the proposed pipeline and utility crossing(s). This drawing will show the following features:

7.3.3.1 Location of the crossing(s), referencing identifiable landmarks including Mileage and Subdivision of the proposed crossing(s) as per CPKC Subdivision naming and Mileage convention. Applicant can obtain the Mileage and Subdivision information from CPKC Utilities group;

7.3.3.2 Pipe centerline, diameter, length, size, limits, thickness and material;

7.3.3.3 Any adjacent structures including but not limited to signals, switches, culverts, other existing underground/buried services including FOTS and vertical distance from BOR;

7.3.3.4 Elevation of surface water in ditches, elevation of the ground water table in all boreholes and the date they were measured;

7.3.3.5 Test pit/borehole locations along with the stratigraphic profile as determined through the geotechnical investigation;

7.3.3.6 Location of jacking or access pits and proposed cut slope angles;

7.3.3.7 Location of the centerline of all tracks;

7.3.3.8 Depth of the top of pipe to the BOR; and

7.3.3.9 Any excavations that encroach on the ZPTL; Indicate ZPTL and distance from ground to the top of pipe.

7.3.3.10 Cross-Sections of perpendicular to the track shall be displayed as viewing in the direction of increasing CPKC mileage; left and right-hand being so determined.

7.4 Geotechnical Investigation Report must be signed and sealed by a locally registered professional Geotechnical Engineer;

7.5 Settlement Monitoring Plan indicating layout and types of settlement monitors to be installed, frequency of measurements, alarm thresholds i.e. “Alert” and “Critical” thresholds, reporting protocol, and immediate actions to take when required. General track movement monitoring guidelines are provided in Appendix C.

7.6 Other Information: This includes information related to additional design and requirements based on the ground conditions and proposed construction. This may include excavation support/shoring, dewatering requirements etc. If required, complete design and relevant drawings will be required.

7.7 Applicant is expected to restore the site to its original condition.

7.8 Proposals for open cut is not a preferred method of installation. This, however, will be assessed on a case by case basis, and prior written approval from CPKC is required for any exceptions.

7.9 Installations using high pressure fluid jetting will not be considered.

7.10 The cost of remediation incurred to CPKC as a result of pipeline and utility crossing(s) construction and installation and related activities will be borne by the crossing(s) applicant. Some of the issues include settlement or soil heave induced by the crossing(s) installation during and after the construction and may be partially offset by the geotechnical retainer fee.

7.11 All pipelines and utilities installed below the highest ground water level predicted will be sealed during construction.

7.12 All pipelines that will or could carry water shall be:

7.12.1 Installed with even bearing throughout its length to limit local settlement; and

7.12.2 Sloped to one end and prevent standing water. Special exemptions will be considered for inverted siphons or other applications requiring level pipes.

8.0 Process 1 – Minimum

8.1 Criteria

The general requirements included in Table 1 in conjunction with the following requirements must be met to obtain approval for a pipeline and utility crossing(s) that qualifies as a Process 1 crossing(s).

Table 2: Process 1 – Minimum

Dimension Criteria	
Outside pipe diameter	Less than 300 mm (12 in.)
Cover between BOR and top of pipe	Greater than 1.5 m (5 ft.) or three pipe diameters whichever is greater.

Adjacent structures including switches and signals	Greater than 10 m (32.8 ft.) centerline.
Depth of pipes outside ZPTL	Refer to SP-TS 2.39 All pipes will be at least 0.91 m (3 ft.) below ground where pipes are not below the ZPTL.
Excavation Criteria	
Excavation close to CPKC track(s)	Jacking/access pits shall be more than 10 m (33 ft.) from the closest track centerline and not encroach on the ZPTL.
Crossing angle	Less than 45 degrees off perpendicular to the track.
Construction Method	
1. Trenchless method[1] 2. Pipe bursting will only be considered where the predicted soil heave is less than 10% of the movement that would result in a change of the FRA or TC track class.	

[1] Trenchless methods include Auger Boring (AB), Pipe Jacking, Pipe Ramming (PR), Horizontal Directional Drilling (HDD) except high pressure fluid jetting method, Microtunnelling (MT) but exclude any type of mining techniques where any stand up time is required before the tunnel support is placed.

8.2 Application Requirements

8.2.1 The applicant will provide documents and drawings containing the information identified in Section 7.0.

8.2.2 Generally, an installation that falls under the minimum review detail level does not require a geotechnical investigation. However, in areas with poor subsurface soil conditions or where failures have occurred with similar pipe crossings, CPKC reserves the right to request a Geotechnical investigation to be conducted in order to proceed with the proposed pipe installation. In situations where a pipe is below the 300mm OD threshold, but the borehole size is 300mm or larger, a Geotechnical investigation is required. Voids between the bore and outside casing are to be filled with non-shrinkable material.

8.2.3 Even if not required by CPKC, a geotechnical investigation may be completed at the discretion of the applicant.

8.3 Application Review and Approval Process

8.3.1 Applicant submits engineering documents to CPKC Utilities.

8.3.2 CPKC Utilities reviews documents to ensure applicable and complete engineering documents are provided.

8.3.3 An assessment is completed by CPKC Utilities to provide decision/approval documentation.

9.0 Process 2 – Intermediate

The Intermediate process pertains to those proposed pipeline/track crossing(s) that exceed the minimum criteria but do not exceed the maximum criteria. The applicant will be required to submit information for review and approval by CPKC Utilities Department or a CPKC approved service provider but may be subjected to additional engineering, monitoring, and construction requirements.

9.1 Criteria

The general requirements included in Table 1 in conjunction with the following requirements must be met to obtain approval for a pipeline and utility crossing(s) that qualifies as a Process 2 crossing(s).

Table 3: Process 2 – Intermediate

Dimension Criteria	
Outside pipe diameter	300 mm (12 in.) to 1500 mm (59 in.)
Cover between BOR and top of pipe	Greater than 1.5 m (5 ft.) or two (2) pipe diameters whichever is greater.

Adjacent structures including switches and signals	Within 2.5 times, cover between BOR and top of pipe.
Depth of pipes outside ZPTL	Less than 0.91 m (3 ft.) burial within ZPTL.
Excavation Criteria	
Excavation close to CPKC track(s)	Excavations or jacking/access pits within 10 m (32.8 ft.) of the closest track centerline.
Crossing angle	More than 45 degrees off perpendicular to the track.
Construction Method	
	1. Trenchless method[1] 2. Pipe bursting will only be considered where the predicted soil heave is less than 10% of the movement that would result in a change of the FRA or TC track class.

[1] Trenchless methods include Auger Boring (AB), Pipe Jacking, Pipe Ramming (PR), Horizontal Directional Drilling (HDD) except high pressure fluid jetting method, Microtunnelling (MT) but exclude any type of mining techniques where any stand up time is required before the tunnel support is placed.

9.2 Application Requirements

9.2.1 Identification of the Geotechnical Engineer of Record (GER). The GER will be responsible for the proposed works on CPKC's ROW from project start up to project closeout including submission of construction summary report and as-built drawing.

9.2.2 Description of the subsurface soil and ground water conditions within and adjacent to CPKC embankment along the proposed pipe/track crossing alignment and to a depth no less than 1.5 times the invert depth below the BOR.

This will consider the impact of silt, fine sand or sand soil, and their relation to the water table and pipe depth. First set of deep monitoring points to be placed on either side of the outside rail at 2m distance off track centerline measured from outside of the rails. Additional deep monitoring points to be placed at the toe of slope and at end points/toes of ZPTL. Signal and fibre locates to be completed before installing any settlement monitoring equipment in the railway right of way.

9.2.3 An estimate of the expected extent and magnitude of ground movement over time based on the proposed pipe installation method will be provided.

9.2.4 A program of ground surface and subsurface (settlement plates) movement monitoring will be implemented. The program must be capable of detecting movement of no less than 50 percent of the movement that would result in a change of the track class as determined by the FRA or TC track safety rules. A real-time remote settlement monitoring system should be used, aiding in reduction of requirements for overnight railway flagging protection when work is paused, but within the ZPTL. Remote settlement monitoring is recommended for all Class 3, 4, and 5 tracks. Manual methods of gathering settlement monitoring readings (such as rod and level) will only be entertained with prior approval.

9.2.4.1 A GIMP (Geotechnical Instrumentation and Monitoring Plan) system will be required if installation is occurring within the zone of potential loading of rail bridge supporting piers or abutments. The instrumentation installed is intended to monitor short and long term embankment performance, along with settlement and stability due to the subsurface site conditions and the nature of the proposed construction activities.

9.2.5 A procedure for notification of the appropriate CPKC personnel in the event that excessive or unexpected settlement occurs. A complete 24 Hour CPKC Emergency contact list, including local personnel and OC will be compiled and in place before any work proceeds within the railway right of way.

9.2.6 A recovery plan will be provided outlining the steps to be implemented in the event of failure (excessive ground loss or settlement / collapse, heaving etc).

9.2.7 Design of de-watering control measures where applicable for the proposed construction method.

9.2.8 Temporary track support system will be required if any of the excavation is closer than 6 m (19.7 feet) from the centre of track and encroaches on the zone of potential track loading. The length of the excavation and an estimated stand-up time of the proposed cut within these limits must be provided and demonstrated to be safe.

9.2.9 A complete description of the proposed construction method.

9.2.10 Confirmation that the proposed construction/installation technique is suited to the site conditions and performance criteria. An assessment of the influence of construction on the track structure including estimated settlement/heave and assessment of risk associated with uncontrolled loss of ground or heaving.

9.2.11 Based on CPKC's review of the conditions, CPKC Geotechnical group may elevate a proposed crossing to Process 3 if deemed necessary.

9.2.12 A qualified independent CPKC approved engineer is required to provide periodic or continuous (at the discretion of CPKC) on-site supervision and document conditions during construction.

9.3 Application Review and Approval Process

9.3.1 Applicant submits engineering documents and utility crossing application to the CPKC Utilities Department.

9.3.2 CPKC Utilities Department reviews documents to check if appropriate and accurate engineering documents have been provided.

9.3.3 CPKC approved Geotechnical service provider to review initially & sign off on behalf of CPKC at applicant's expense. CPKC Geotechnical to provide final geotechnical approval.

9.3.4 CPKC Structural Engineering Group may have to provide structural approval, if required.

9.3.5 CPKC Utilities Department – To provide final decision or approvals.

10.0 Process 3 – Detailed

Process 3 will be applicable to those crossing(s) applications that do not meet the conditions of Process 2. In this case, expert engineering submissions are required, along with additional work such as dewatering as well as monitoring by on site engineering consultants during construction.

10.1 Criteria

The general requirements included in Table 1 in conjunction with the following requirements must be met to obtain approval for a pipeline crossing(s) that qualifies as a Process 3 crossing.

Table 4: Process 3 – Detailed

Dimension Criteria	
Outside pipe diameter	Greater than 1500 mm (59 in.)
Cover between BOR and top of pipe	Less than 1.5 m (5 ft.) or two (2) pipe diameters.
Adjacent structures, switches and signals	Within 2.5 times, cover between BOR and top of pipe.
Depth of pipes outside ZPTL	Less than 0.91 m (3 ft.) burial within ZPTL.
Excavation close to CPKC track(s)	Excavations or jacking/access pits within 10 m (33 ft.) of the closest track centerline.
Excavation Criteria	
Excavation close to CPKC track(s)	Excavations or jacking/access pits within 10 m (30 ft) of the closest track centerline.
Crossing angle	More than 45 degrees off perpendicular to the track.
Construction Method	

All methods considered
Ground conditions, complex installation method, and/or the complexity of the project warrant that specialist-engineering personnel review the design and or construction of the pipe/track crossing(s).

10.2 Application Requirements

10.2.1 The applicant will meet the requirement outlined in Process 2 - Section 9.2.

10.2.2 The applicant will provide resources for CPKC to retain CPKC approved service provider(s) or experts(s) to assess and review the application and advise CPKC on the impact of the applicant's proposal on CPKC ROW.

10.3 Application Process and Approval Process

10.3.1 Applicant submits engineering documents to CPKC Utilities. All applications will be processed as per the procedure outlined in Section 9.3.

11.0 Pre-Construction Meeting Requirement

Prior to commencement of any work within CPKC property/ROW, the Geotechnical Engineer of Record (GER) or their designate shall arrange a pre-construction meeting at least thirty days before with all stakeholders to discuss project and construction details including work description, construction methods and schedule, restrictions, safety, hours/days of work, start time, Daily Reporting & other CPKC requirements and agreed upon Protocols governing Extreme Weather/Rainfall Warning Alerts issued from Local/National weather offices. This may mean that drilling operations ceases until these Alerts are no longer in effect. It is the responsibility of the GER or their designate to ensure that flagging protection has been arranged for the duration of the project, all construction oversight and track settlement monitoring review has been arranged with CPKC approved service provider and that the expectations have been clearly communicated before construction commences.

12.0 Daily Inspection & Reporting during Construction

This section is applicable to Process Levels 2 and 3 application proposals. The agreement holder or applicant will identify a Geotechnical Engineer of Record (GER) responsible for the complete work and installation of proposed crossing/excavation within CPKC ROW from start to finish. The Geotechnical Engineer of Record may assign a competent/trained person to act as Site

Inspector/Engineer who will be present onsite during the full duration of the bore or any other ground disturbance activity within railway operating corridor, unless, otherwise directed by CPKC Utilities Supervisor. Depending on the complexity of the installation and or field issues encountered during the installation that may adversely impact CPKC Infrastructure, CPKC may, at their discretion, assign a full time Geotech Monitor, of their choice, to be on site, at the Applicant's expense.

CPKC flagger or assigned representative must be present at all times when working or drilling within CPKC property or rail operating corridor. No movement of pipe within the ROW or ZPTL is permitted without the presence of a CPKC flagger unless prior written approval from CPKC for an exemption has been provided. The Site Inspector/Engineer must have the required training, experience and understanding of the site conditions, proposed design, and construction methodology to make sound engineering judgement and decisions, and reports during the course of the work.

The Site Inspector/Engineer must ensure that the work is being carried out in accordance with the approved designs, permits and procedures, and/or relevant specifications. The Site Inspector/Engineer must immediately report any issues encountered during construction work and could have an impact on CPKC assets and its operations. Some examples include instability or potential of instability of the embankment or potential ground settlements either future or immediate.

Any concerns about the imminent stability of the grade shall immediately be escalated to CPKC Flagger or representative in order to protect against train operations. In addition, refer to CPKC 24 Hour Emergency Contact list to use in case of emergency. The concerns shall also be escalated to the GER and CPKC Utilities supervisor so immediate remediation plans can be implemented.

The Site Inspector/Engineer will provide a daily report to CPKC approved service provider, copying CPKC Utilities supervisor, CPKC's Director Geotechnical Engineering and the GER, outlining the progress during the day, any deviations from the original plans, any unexpected ground conditions, or any issues that were encountered during the construction. The report shall also contain relevant information that assures CPKC that the field activities are being monitored and documented to ensure that the installation is proceeding in accordance with approved plans and no unexpected conditions/issues are expected. Some examples of relevant information examples include some of the following information:

- A quantitative estimation of amount of material removed versus theoretical material;
- Auger location - Location of both, the leading edge of the pipe and the location of the leading edge of the auger should be documented;

- A description of the progress and any observations or issues encountered during the pipe installation including geologic conditions, change in material composition, characteristics, etc.

The daily report will also include all settlement monitoring data, along with any pertinent photos. If applicable, this report will also make notes and highlight any measures taken for “out of compliance” practice or when conditions requiring attention are expected or encountered. See Appendix B for a Sample Report.

A mid-day report should also be submitted by 13:00 local time each day until installation clears the railway right of way and no further movement is occurring due to the installation activities. This requirement can be reviewed and waived if agreed upon by all parties during the preconstruction meeting.

Upon completion of the construction and installation of pipeline and utility crossing(s), the GER will provide a final sealed and stamped letter/construction report to CPKC approved service provider with a copy to CPKC Utilities supervisor confirming that the work has been completed in accordance with the approved plans and procedures. If there are any deviations from the approved plans/procedures, these must be noted in the final letter/report. As-built stamped drawings are to be submitted to the CPKC Utilities Department along with final settlement data collected and correspondence.

All costs associated with above mentioned i.e. complete geotechnical review, track settlement monitoring, flagging and construction oversight provided CPKC approved service provider will be borne by the applicant.
A contract between CPKC approved service provider(s) and the applicant must be place before proceeding with this work proposal.

13.0 Review Steps

The following is a checklist of steps that will be completed to ensure that the appropriate level of care has been taken for Process 2 and 3 pipe crossings below the track.

Table 5 – Review Steps

No.	Step	Action/Review by

No.	Step	Action/Review by
13.1	Submission of crossing(s) proposal by applicant including details of the crossing(s) specification and potential construction method(s) to CPKC Utilities.	Applicant
13.2	Review of the proposal as per this protocol to determine what level of geotechnical engineering and review is required.	CPKC Utilities
13.3	Designation of review i.e. CPKC approved service provider. (ASP)	CPKC Utilities
13.4	Identification of the Applicant's Geotechnical Engineer of Record.	CPKC Geotech Engineering/ASP
13.5	Assessment of adequacy of the geotechnical investigation and other required information.	CPKC Geotech Engineering/ASP
13.6	Applicant's geotechnical engineer determines that the proposed construction/installation method will not cause settlement of the CPKC track or structures.	CPKC Geotech Engineering/ASP
13.7	Settlement monitoring program, if required and developed by the applicant's geotechnical engineer.	CPKC Geotech Engineering/ASP
13.8	Once a contractor has been selected, the Geotechnical Engineer of Record (GER) will review the shop drawings submitted by the contractor or the sub-contractor(s) to determine if the tunnelling and dewatering (if required) method proposed could cause track settlement.	CPKC Geotech Engineering/ASP
13.9	Applicant will provide CPKC with written documentation of who will be completing the onsite review of the contractor's construction practice and the specifics of the assignment.	CPKC Geotech Engineering/ASP
13.10	Applicant will enlist the services of a Geotechnical Engineer of Record(GER) with the responsibility for inspection of the tunnelling contractor's work. They will also assure that adequate measures are in place to minimize the potential for track settlement. The intention is to assign an appropriate group with the task of assuring that actions undertaken by the contractor do not endanger the track structure because of ground loss during tunneling which may affect CPKC Train operations.	CPKC Geotech Engineering/ASP
13.11	An emergency response will be developed and posted on site and will reside with key personnel.	CPKC Geotech Engineering/ASP
13.12	A contingency plan will be prepared and submitted by Tunneling contractor prior to start of construction, identifying tasks/activities that can be completed within hours to get track back in service, if	CPKC Geotech Engineering/ASP

No.	Step	Action/Review by
	significant track settlement is experienced.	
13.13	24 Hour Emergency Contact List to be provided prior to commencement of construction.	CPKC Utilities

14.0 Abandoned Pipe/Track Crossing(s)

In the event that an existing installation is abandoned or a proposed crossing(s) is abandoned during construction, all potential hazards to CPKC property must be removed or abated. This may be achieved by removal of any buried pipes and the backfill and compaction of any excavations. Alternately, upon approval of the CPKC Geotechnical group any voids within ground may be backfilled with non-shrinkable fill, or pressured grout sufficient to prevent future sloughing or track settlement. Any buried material (wood or metal) that could increase or decrease volume over time due to chemical reaction (oxidation) or decomposition must be removed or stabilized to the satisfaction of CPKC.

Appendix A

SAMPLE DAILY REPORT AND SETTLEMENT REPORT

SAMPLE DAILY REPORT

PROJECT INFORMATION

Project Name:		Date:	July 11, 2019
Location:		Contractor:	
Client:		Site Rep:	

DAILY SUMMARY

Excavation Details:

[i.e., depth, sideslopes, trench boxes, sloughing, etc.]

No additional casing installed today. No sloughing of CPR embankment noted.
See attached sketch.

Construction Summary:

[i.e., soil type, issues, etc.]

Contractor attaching shoes (wedges) to the casing exterior near the casing head behind the gravel plug (approx. station 0+031). One shoe on track west side of casing approx. 250 mm X 300 mm X 50 mm(deep outside of casing). One shoe on bottom of casing approx. 275 mm X 300 mm X 25 mm(deep outside of casing).

Cumulative Settlement Movement (mm): ☒ Minimal Movement (<8) ☐ Level 1 – Warning (≥8 to <16) ☐ Level 2 – Critical (≥16)

Compliance with Design: ☐ Yes ☒ No If No, discuss below

Issues with Installation: ☒ Yes ☐ No If Yes, discuss below

Other Notes, if any:

Casing was out of alignment by approximately 40 mm track west and 25 mm lower than designed location at about station 0+031. Contractor attaching shoes (wedges) to the casing exterior near the casing head behind the gravel plug (approx. station 0+031) for alignment correction.

Prepared By: Reviewed By:

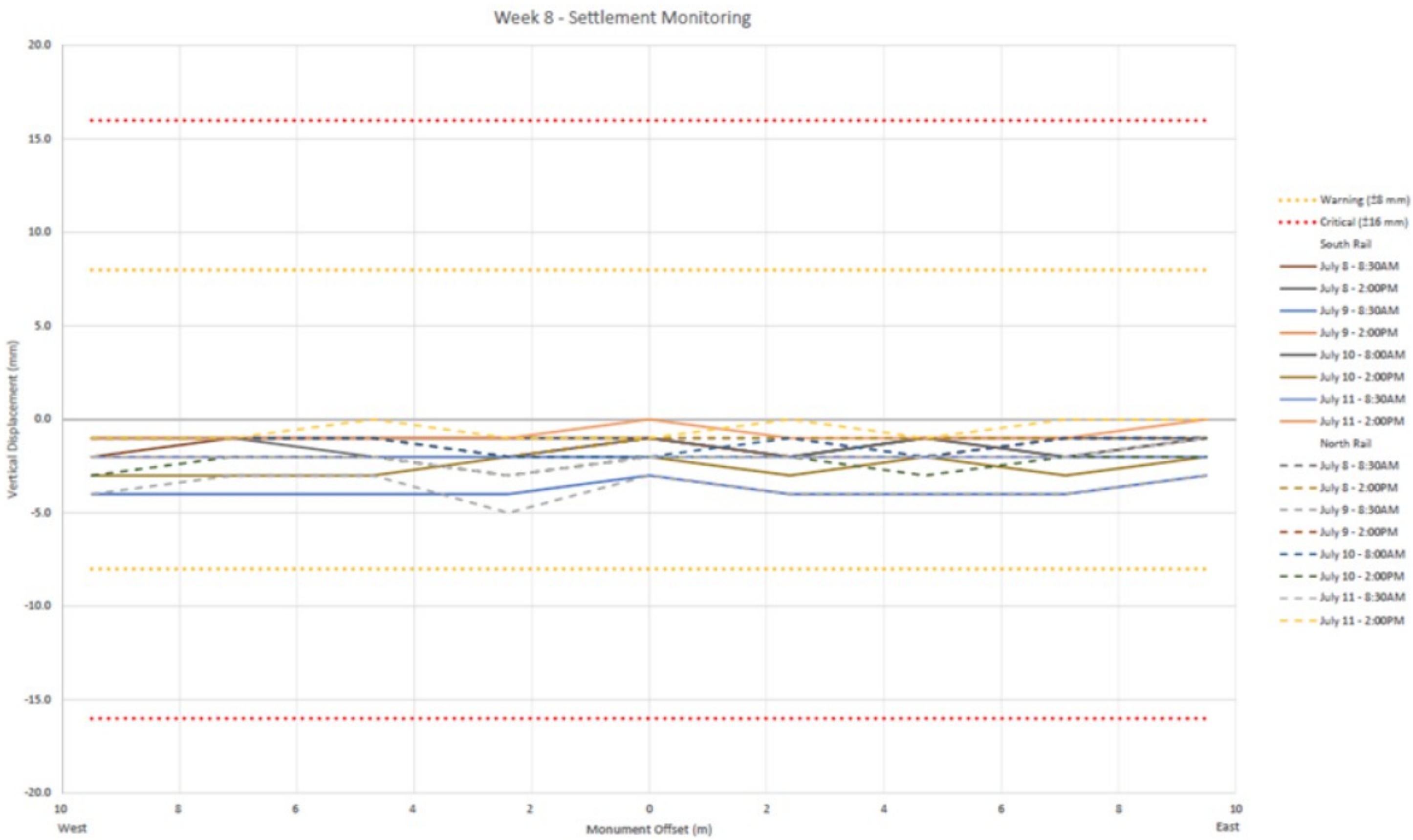
SAMPLE DAILY SETTLEMENT MONITORING REPORT

SETTLEMENT MONITORING

Surveyor: D. Makowsky (Stantec) **Date:** July 11, 2019 **CPR Flagman:** Kashpal Jaswal
Weather: Sunny **Temperature:** 16°C at 8:30 am; 27°C at 2:00 pm

*Survey locations along the top of the rails. **Baseline elevation is average of 2 surveys on June 12, 2019. ***Positive numbers correspond to heave and negative numbers correspond to settlement

Rail Location	Location Description and Monument ID*	Baseline Elevation (m)**	Reading 1			Reading 2		
			Time	Elevation (m)	Cumulative Movement (mm)***	Time	Elevation (m)	Cumulative Movement (mm)***
South Rail	9.45 m East (S9)	1034.587	8:30 am	1034.585	-2	2:00 pm	1034.587	0
	7.08 m East (S8)	1034.593		1034.591	-2		1034.592	-1
	4.72 m East (S7)	1034.597		1034.595	-2		1034.596	-1
	2.36 m East (S6)	1034.601		1034.599	-2		1034.600	-1
	Centerline (S5)	1034.605		1034.603	-2		1034.605	0
	2.36 m West (S4)	1034.612		1034.610	-2		1034.611	-1
	4.75 m West (S3)	1034.618		1034.616	-2		1034.617	-1
	7.08 m West (S2)	1034.622		1034.620	-2		1034.621	-1
	9.45 m West (S1)	1034.626		1034.624	-2		1034.625	-1
North Rail	9.45 m West (N1)	1034.624		1034.622	-2		1034.623	-1
	7.08 m West (N2)	1034.621		1034.619	-2		1034.620	-1
	4.75 m West (N3)	1034.617		1034.615	-2		1034.617	0
	2.36 m West (N4)	1034.612		1034.609	-3		1034.611	-1
	Centerline (N5)	1034.604		1034.602	-2		1034.603	-1
	2.36 m East (N6)	1034.599		1034.597	-2		1034.599	0
	4.75 m East (N7)	1034.597		1034.595	-2		1034.596	-1
	7.08 m East (N8)	1034.590		1034.588	-2		1034.590	0
	9.45 m East (N9)	1034.586		1034.585	-1		1034.586	0


Week 8 - Settlement Monitoring
Vertical Displacement (mm)
20.0
15.0
10.0
5.0
0.0
-5.0
-10.0
-15.0
-20.0
10
West
8
6
4
2
0
2
4
6
8
10
East
Monument Offset (m)
Warning (±8 mm)
Critical (±16 mm)
South Rail
July 8 - 8:30AM
July 8 - 2:00PM
July 9 - 8:30AM
July 9 - 2:00PM
July 10 - 8:00AM
July 10 - 2:00PM
July 11 - 8:30AM
July 11 - 2:00PM
North Rail
July 8 - 8:30AM
July 8 - 2:00PM
July 9 - 8:30AM
July 9 - 2:00PM
July 10 - 8:00AM
July 10 - 2:00PM
July 11 - 8:30AM
July 11 - 2:00PM

SAMPLE TRENCHLESS INSTALLATION MONITORING REPORT

TRENCHLESS INSTALLATION MONITORING

Equipment:	Air Hammer	**Operator:**	Jorden Calgary Tunneling
Casing Pipe Diameter:	750 mm	16 mm overcut on casing head	
Date:	July 11, 2019	2m plug at casing head during installation	

Date	Segment No.	Segment Length (m)	Station Interval along the Face of Installation	Length of Pipe Installed (m)	Distance of Head to CL Track (S or N) (m)	Soils Condition/Description
July 7, 2019	1	6.0	0+015	6.0	18 m N	Gravel, sandy, some silt, trace clay. Dry
July 8, 2019	2	6.0	0+021	6.0	12 m N	Gravel, sandy, some silt, trace clay. Dry
July 9, 2019	3	6.0	0+027	6.0	6 m N	Gravel, sandy, some silt. Damp
July 9, 2019	4	6.0	0+033	6.0	0 m (CL Track)	Gravel, sandy, some silt. Damp
July 10, 2019 July 11, 2019						No casing installed, contractor attaching shoes (wedges) to casing at station 0+031

N1
N2
N3
N4
N5
N6
N7
N8
N9
S1
S2
S3
S4
S5
S6
S7
S8
S9
9.45 m
7.08 m
4.72 m
2.36 m
UPDATED CL OF WATERMAIN
CL OF 450 WATERMAIN (ORIGINAL)
26A STREET SE
LEGEND
SURFACE SETTLEMENT MONITORING POINT (PREVIOUS)
UPDATED SURFACE SETTLEMENT MONITORING POINT (JUNE 12)
0
20m
LOCATION OF SETTLEMENT MONITORING MONUMENTS
PROJECT NO.
C12103500PKG-022.4
DWN
LCH
CKD
KH
REV
0
OFFICE
TT Cal
DATE
May 2019
Figure

Appendix B

TRACK MOVEMENT MONITORING GUIDELINES FOR TRENCHLESS PIPE INSTALLATION

Track Movement Monitoring Guidelines for Trenchless Pipe and Utility Crossing(s) Installation under Railway Tracks

The monitoring of track settlement should be carried out by means of surface and subsurface settlement points. The intent of subsurface settlement points is to measure voids created just in the vicinity and above the pipe during construction in order to predict the potential movement of overlying CPKC tracks.

The settlement point essentially consists of a small diameter pipe anchored at the bottom of a vertical borehole and an outer casing to isolate the pipe from down drag forces caused by settlement of soil above the anchor (see Figure B). The subsurface settlement points would be installed to 1 m above the crown of the casing profile. The total number of subsurface settlement points within CPKC Right-of-Way (ROW) along the axis of the proposed pipe crossing(s) would be installed as per the configuration shown in Figure A – Sample Surface and Subsurface Settlement Monitoring Layout.

Surface points installed directly along the base of both rails at a spacing of 9.45 m (31 ft.) over the projected settlement trough would be used to monitor differential transversal elevation between both rails. The total number of surface settlement points within CPKC ROW would be installed as per the configuration shown in Figure A – Sample Surface and Subsurface Settlement Monitoring Layout. These points shall be monitored simultaneously with the subsurface settlement points that would act as a precursor to potential surface movement during pipe installation.

Once the installation is complete, a monitoring program of all points is to be carried out in accordance with the following instructions:

1. Monitoring should start before the excavation of the pits and pipe installation begins and readings should be taken at least twice per day for no less than two days. This is required to establish a reliable methodology and demonstrate the accuracy to be achieved.

2. Monitoring should proceed through the construction period and should be completed:

    1) For branch lines/lines with low traffic - (Class 1-2 Track) - At least twice daily.

    2) For main lines and heavy traffic lines – (Class 3-5 Track) Every 2 hours and before and after each train, whichever provides the most number of readings while the boring operation is within the ZPTL (Zone of Potential Track Loading).

3. Monitoring should continue for at least 3 days after the completion of construction.

4. If there is any loss of ground during pipe installation, any reason to believe settlement may be delayed or any settlement is identified during the installation of pipe or subsequent monitoring period, the monitoring must be continued until the applicant's Geotechnical Engineer of Record deems it is safe to discontinue such monitoring. This must be approved by CPKC Geotechnical Engineering group or CPKC approved service provider reviewing the monitoring results.

Monitoring measurements should be taken with sufficient frequency (as noted above) to capture the unexpected performance at the earliest possible stage and be evaluated in a timely manner. Additional measures will be proposed should this monitoring protocol be considered insufficient based on the ground conditions or installation process. Track survey preference would be for survey shots to be taken remotely (i.e. off CPKC property) and without the requirement of a CPKC Flagger or representative presence on site.

Two alarm levels are proposed:-

Level 1:

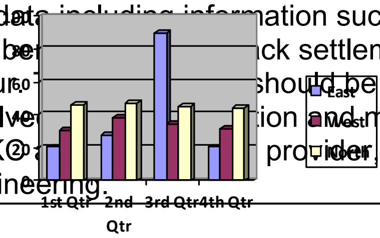
ALERT – (Review Threshold) must be indicated on the field memo/report when a settlement of **50 (%)** of the critical monitoring threshold is obtained from the subsurface and/or surface settlement points. A survey of the surface points will then be carried out and work will be authorized to continue if no movement of the subsurface point has been measured from the previous reading. If movement of the rails is recorded, monitoring will be continued until rail movement is stopped. At this point, the drilling work will then be authorized to continue. See Figure C – Track Settlement Monitoring Review and Alert Threshold for Threshold values per Class of Track designation. Please contact CPKC Utilities Supervisor to obtain Class of Track designation pertaining to the proposed Utility Crossing location. CPKC Geotechnical Engineer/Utility Supervisor should be called to discuss these findings in order to discuss next steps.

Level 2:

CRITICAL – (Stop Work) - Installation **must** come to an immediate stop if monitoring points trigger Critical levels.
Above information must be indicated on the field memo/report when a settlement of specified monitoring threshold is obtained from the subsurface settlement point. A survey of the surface points will then be carried out and work will be authorized to continue if no movement is measured for at least two (2) readings taken 12 hours apart. If movement of the rails is recorded, monitoring will be continued until movement is stopped and the applicant has submitted a new pipe installation procedure. This procedure must be reviewed and approved by CPKC Geotechnical Engineering group or CPKC approved service provider reviewing the monitoring results.

The applicant and their Geotechnical Engineer of Record are responsible for ensuring that track settlement does not occur and for notifying CPKC Roadmaster or their designate, as indicated on the 24 Hour Emergency Contact List, should unforeseeable track settlement occur or be expected.

The above guidelines do not relieve the applicant and their engineer(s) of this responsibility. The applicant or their engineer(s) shall provide the settlement information and their interpretation of the data including information such as. no track settlement, deep settlement etc., a quantitative number [illegible] track settlement has occurred, is likely to occur and when it is likely to occur. [illegible] should be provided in easily understandable terms for all parties involved [illegible] tion and monitoring and should be directed to local CPKC Roadmaster, CPKC [illegible] provider, Supervisor – CPKC Utilities and Director of Geotechnical Engineering.



SAMPLE TRACK SETTLEMENT MONITORING DRAWING

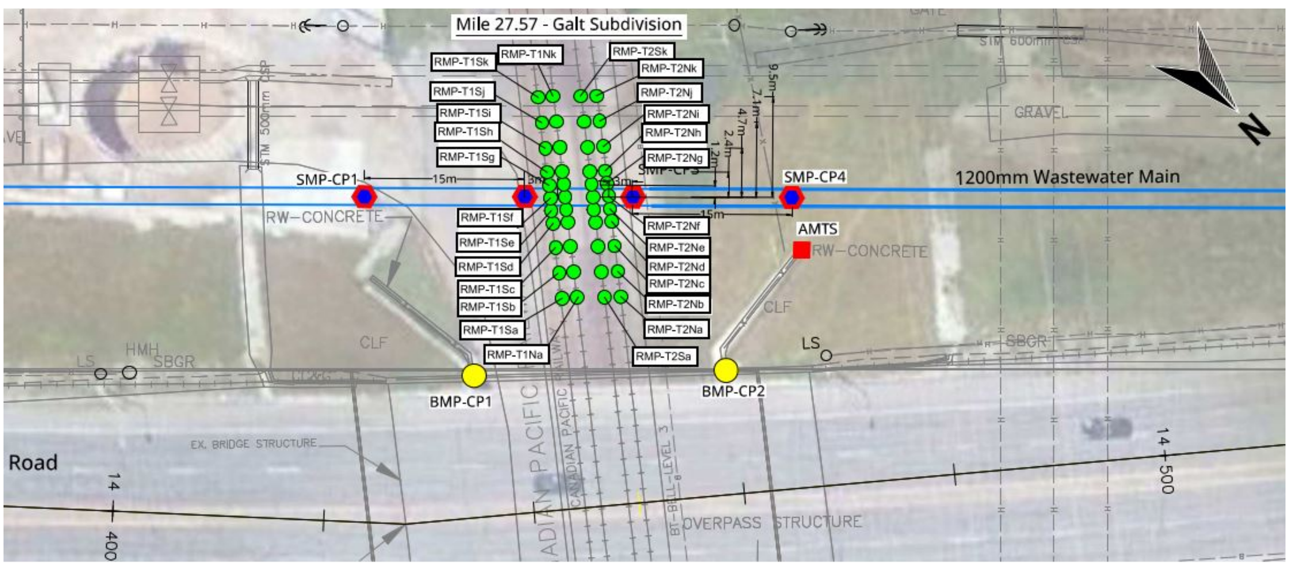


Figure A - Sample Surface and Subsurface Settlement Monitoring Layout

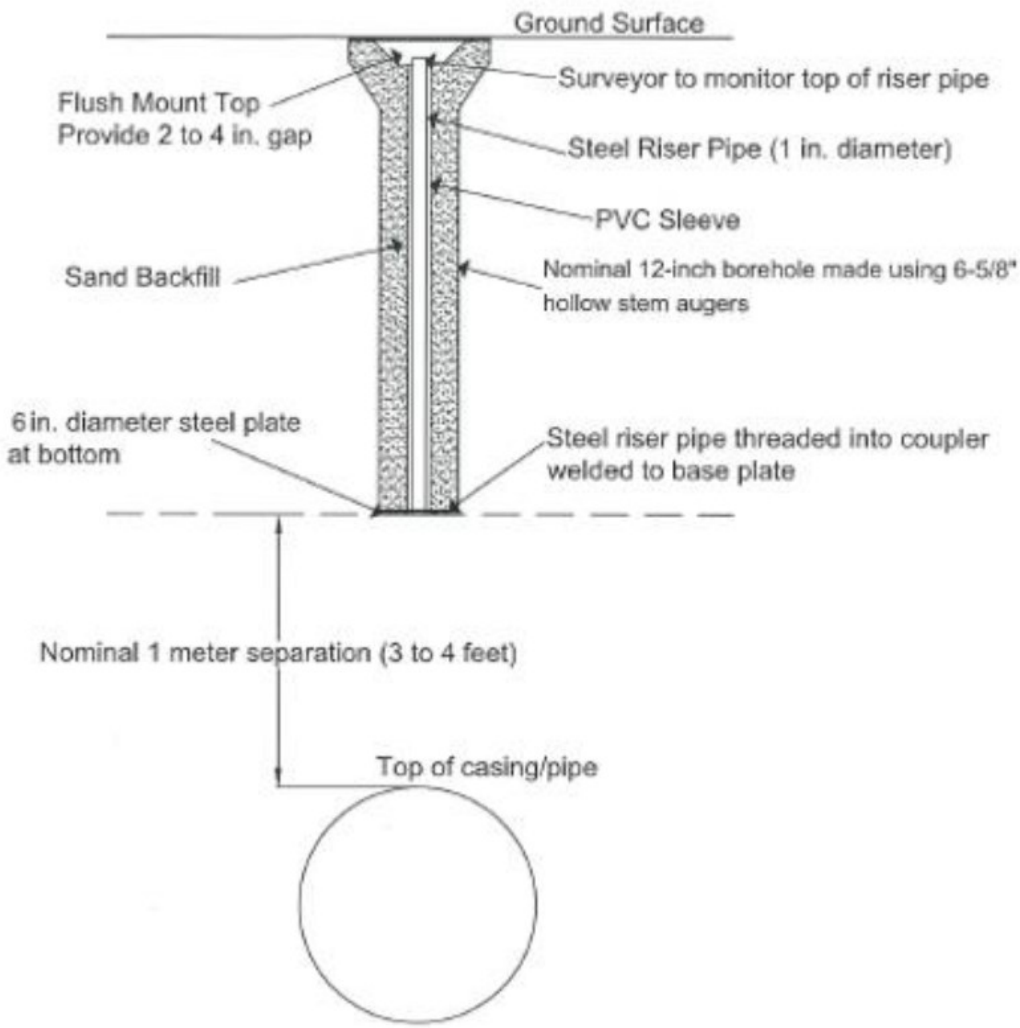


Figure B – Track Settlement Monitoring Critical and Alert Thresholds (Surface and Subsurface)

Class of track	Critical Threshold	Alert Threshold
1	22 mm	11 mm
2	22 mm	11 mm
3	19 mm	10 mm
4	16 mm	8 mm
5	13 mm	6 mm
6	10 mm	5 mm

Note – All above numbers are maximum values

Class of Track

TRACK CLASSES

Class	Freight Train Speed	Passenger Train Speed
1	10 MPH	15MPH
2	25 MPH	30 MPH
3	40 MPH	60 MPH
4	60 MPH	80 MPH
5	80 MPH	95 MPH* 90 MPH **
*Denotes for LRC trains – 100 MPH	** - Applies to US only	Note – Numbers above are maximum values

Appendix C

ADDITIONAL NOTES & INSTALLATION REQUIREMENTS

1. Many of CPKC's properties contain buried parallel fibre optic networks. CPKC will supply the appropriate 1-800 numbers to call to ensure the protection of these fibre lines when crossing. The applicant must arrange with the various fibre maintenance providers for the proper hand digging and exposure of the fibre cable prior to commencing construction. No pipelines or cable crossings are to be installed at less than 1 vertical meter above or below the fibre cables, and no buried parallel occupancies, poles or anchors are to be located within 3 horizontal meters of the fibre optic cables.

2. In absolutely no instance is the utility to be installed without receiving prior approval from CPKC and arranging with the Utilities group for track protection. Any contractors entering the property prior to making these arrangements or without the presence of a CPKC representative will be subject to immediate and lengthy work stoppages by the railway.

CASING AND INSTALLATION OF LONGITUDINAL PIPES, AND PIPES IN CLOSE PROXIMITY TO BRIDGES AND IMPORTANT STRUCTURES:

The AREMA Specifications address pipeline installation in proximity to railway bridges with the following clauses:

1. Pipelines shall be located, where practicable, to cross tracks at approximately right angles thereto but preferably at not less than 45 degrees and shall not be placed within culverts nor under railway bridges where there is a likelihood of restricting the area required for the purposes for which the bridges or culverts were built, or of endangering the foundations.

2. Pipelines laid longitudinally on railway rights-of-way shall be located as far as practicable from any tracks or other important structures. If located within 25 feet (7.62M) of the centerline of any track or where there is danger of damage from leakage to any bridge, building or other important structure, the carrier pipe shall be encased or of special design as approved by the engineer.

Whereas the AREMA specifications require that longitudinal pipelines, and those in proximity to a bridge or other important structure be encased if within 7.62 M of the track or structure, or of special design as approved by the engineer, should the pipeline be encased;

1. CPKC requires that the length of the casing pipe adjacent to a track shall be for the full length of pipe falling within the 7.62 M distance from the track, and

2. If adjacent to a bridge or structure, the casing pipe must extend to the point where the end of the casing pipe is a minimum of 7.62 M beyond the nearest points of the structure or bridge foundation.

In all cases, the design engineer must be confident that the depth, ground conditions and method of installation used will not in any way interfere with the integrity of the track bed and/or adjacent foundations and they must also provide CPKC with a stamped design plan or report, detailing the installation methodology to be used.

The following tables may be used for water, sewer, steam and non-flammable substances, and are Metric versions of the tables contained in the AREMA manual.

Minimum Wall Thickness for Steel Casing Pipe for E80 Loading:

Diameter (mm) less than or equal to	When coated or cathodically protected Nominal Thickness (mm)	When **not** coated or cathodically protected Nominal Thickness (mm)
324	4.77	4.77
356	4.77	6.35
406	5.59	7.14
457	6.35	7.92
559	7.14	8.74
610	7.92	9.53
660	8.74	10.31
711	9.53	11.13
762	10.31	11.91
813	11.13	12.70
914	11.91	13.49
965	12.70	14.27
1016	13.49	15.09
1067	14.27	15.88
1168	15.09	16.66
1219	15.88	17.48
1270	16.66	18.26
1321	17.48	19.05
1372	18.26	19.84
1473	19.05	20.62
1524	19.84	21.44
1575	20.62	22.23
1626	21.44	23.01
1727	22.23	23.83
1778	23.01	24.61
1829	23.83	25.40

Note: The length of steel casing pipe in this table and the steel carrier pipe in the following table must be as per CPKC Specification 2.39 Appendix A.

The inside diameter of the casing pipe must be at least 50.8 mm larger than the outside diameter of the carrier pipe if the carrier pipe is 152.4 mm or less. For all carrier pipes with outside diameters in excess of 152.4 mm, the inside diameter of the casing pipe must be at least 101.6 mm larger than the outside diameter of the carrier pipe.

The following Tables give the minimum thickness for steel carrier pipe for E80 loading.

Note: The length of the steel carrier pipe in these tables must be as per CPKC Specification 2.39 Appendix A. Additionally, all carrier pipes that are not provided with cathodic protection, (impressed current or sacrificial anode) must be a minimum of 1.59 mm thicker than shown in these tables.

Dia.	Minimum Yield Strength (mPa) > =					Minimum Yield Strength (mPa) > =				
(mm)	241	290	358	414	483	241	290	358	414	483
	MAOP < = 689 kPa					MAOP < = 1379 kPa				
<=457.2	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77	4.77
508	5.56	5.56	5.56	5.56	5.56	5.56	5.56	5.56	5.56	5.56
558.8	5.74	5.74	5.74	5.74	5.74	5.74	5.74	5.74	5.74	5.74
609.6	6.35	6.35	6.35	6.35	6.35	6.35	6.35	6.35	6.35	6.35
660.4	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14
711.2	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14
762	7.92	7.92	7.92	7.92	7.92	7.92	7.92	7.92	7.92	7.92
812.8	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74
863.6	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74	8.74
914.4	9.53	9.53	9.53	9.53	9.53	I0.3I	9.53	9.53	9.53	9.53
965.2	10.31	10.31	10.31	10.31	10.31	10.31	10.31	10.31	10.31	10.31
1016	10.31	10.31	10.31	10.31	10.31	11.91	10.31	10.31	10.31	10.31
1066.8	10.31	10.31	10.31	10.31	10.31	12.7	10.31	10.31	10.31	10.31

Dia.	Minimum Yield Strength (mPa) > =					Minimum Yield Strength (mPa) > =				
(mm)	241	290	358	414	483	241	290	358	414	483
	MAOP < = 11721 kPa					MAOP < = 12411 kPa				
<= 101.6	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78	4.78
114.3	4.78	4.78	4.78	4.78	4.78	5.16	4.78	4.78	4.78	4.78
141.3	6.55	4.78	4.78	4.78	4.78	6.55	5.56	4.78	4.78	4.78
168.3	7.11	6.35	4.78	4.78	4.78	7.92	6.35	5.56	4.78	4.78
219.1	9.53	7.92	6.35	5.56	4.78	9.53	7.92	6.35	5.56	4.78
273.I	11.13	9.27	7.92	6.5	5.56	11.91	10.31	7.92	7.09	6.35
323.9	13.49	11.13	9.53	7.92	7.14	14.27	11.91	9.53	8.74	7.I4
355.6	15.09	12.7	10.31	8.74	7.92	15.88	12.7	10.31	9.53	7.92
406.4	16.66	14.27	11.13	10.31	8.74	17.48	15.09	11.91	10.31	8.74
457.2	19.05	15.88	12.7	11.13	9.53	19.84	16.66	13.49	11.91	10.31
508	20.62	17.48	14.27	12.7	10.31	22.23	18.26	15.09	12.7	11.13
558.8	23.01	19.05	15.88	13.49	11.91	24.61	19.84	16.66	14.27	12.7
609.6	25.4	20.62	16.66	15.09	12.7	26.19	22.23	18.26	15.88	13.49
660.4	26.97	23.01	18.26	15.88	13.49	28.58	23.83	19.05	16.66	14.27
711.2	29.36	24.61	19.84	17.48	15.09	30.96	25.4	20.62	18.26	15.88
1016	36.53	30.96	25.4	22.23	19.05	38.89	32.54	26.97	23.01	19.84
1066.8	38.89	32.54	26.97	23.83	19.84	---	34.14	27.79	24.61	21.44

REQUIREMENTS FOR THE DESIGN OF STEEL CULVERTS CARRYING RAILWAY TRAFFIC

1. Design Specifications

AREMA Manual of Recommended Practice, Chapter 1: Part 4: Culverts, latest edition.

2. Type of Construction, Materials, Structural Design and Installation

Culverts may be constructed with corrugated steel pipe (CSP, shop fabricated); structural plate corrugated steel pipe (SPCSP, field fabricated) or steel pipe (bored or jacked).

CSP installations shall be in accordance with CPKC Standard Plans B-1-4950-1 (Canada) or B-1- 4950-2 (United States). These standard plans outline material, structural and installation requirements for CSP installations up to 1800 mm (6'-0") in diameter.

SPCSP installations, and installations using materials other than corrugated steel, require specific design and plans relating to material, structural and installation requirements to be prepared by a qualified professional engineer.

Steel pipe installations shall be in accordance with Table 4.9 "Least Nominal Wall Thickness for Steel Casing Pipe in Cased Crossings and Carrier Pipe in Uncased Crossings" in C.S.A. Standard Z662, latest edition, as amended by the Transport Canada "Standards Respecting Pipeline Crossings Under Railways" (originally invoked May 10, 2001); or as otherwise required by the proposed method of installation.

3. Hydraulic Design

Many culverts, based on history of the installation and experience of local officers, are replaced in- kind without need of a hydrological assessment. However, a hydrological assessment is required for new culvert installations, installations where a change in watercourse conditions has occurred, or where required by regulatory authorities. Where a hydrological assessment is performed, culvert requirements shall be determined in accordance with the following hydraulic criteria:

1. Culverts under main line tracks shall de designed for the following, whichever is greatest;

The 50-year flood with culvert pipes flowing no greater than 2/3 full (head to depth ratio less than 0.67); or

The 100-year flood with culvert pipes flowing no greater than full (head to depth ratio less than 1.00), where culvert cover is not less than 1500 mm (5'-0"). Where culvert cover is less than 1500 mm (5'-0") culverts shall be designed for the 100-year flood frequency flow with culvert pipes flowing no greater than 2/3 full (head to depth ratio less than 0.67).

2. Culverts under secondary and branch lines shall be designed for the following, whichever is greatest;

The 50-year flood with culvert pipes flowing no greater than full (head to depth ratio less than 1.00); or

The 100-year flood with culvert pipes flowing with a headwater depth no greater than 50% of the diameter of the pipes above the top of pipe (head to depth ratio less than I.50). However the headwater depth shall not be less than I metre (3 feet) below base-of-rail.

The following table lists the minimum requirements for round CMP pipes used as casing pipes for water, sewer, steam and non-flammable materials.

Culvert Size (mm) (Inside Dia.)	Corrugation Profile (mm)	Min. Depth of Cover (base of rail to top of pipe) (mm)	Max. Depth of Cover (m) (base of rail to top of pipe			
			Specified Wall Thickness (mm)			
			1.6	2	2.8	3.5
300	68 x 13	1100	15.2	15.2		
380	68 x 13	1100	12.2	12.2		
460	68 x 13	1100	9.1	9.1	16.8	
530	68 x 13	1100	7.6	7.6	13.7	
600	68 x 13	1100	7.6	7.6	12.2	13.7
760	68 x 13	1100			9.1	10.7

In all cases where inside diameters exceed 760 mm, CMP casing pipes shall be designed as per CPKC standard plan B-1-4950-1.

Culverts must be zinc or aluminum coated. Additional coatings and couplings shall be provided as per CPKC standard plan B-1-4950-1.

Some supplementary information contained in the AREMA specification, regarding pipeline (not including Gas and Oil pipelines) and casing pipes for wire crossings of the Railway is as follows:

Calculation for Cooper E80 Loading for pipelines in pounds per square foot

T_{E80} = Total E80 Load in pounds per square foot
L_L = Live Load in pounds per square foot
I_P = Impact Loading Percentage
L_D = Dead Load in pounds per square foot
D = Lateral Live Load Distribution Length in feet
H = Depth of cover in Feet
W = Weight of overburden in pounds per cubic foot.

$T_{E80} = L_L*(1.0 + I_P) + L_D$

$L_L = 80000 / (5 * D)$

$L_D = W * H + 200 / D$

$I_P = (10 - H) * .04$ Negative results equate to zero.

$D = (8.5 + H)$

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