



FINAL REPORT

Foundation Investigation and Design Report

Detailed Design Proposed Arnprior Maintenance Patrol Yard

Location 12 - Highway 417 and Ottawa Road 29,

Arnprior, Ontario

MTO Agreement No. 4020-E-0012-32

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PART A

Foundation Investigation Report

Detailed Design Proposed
Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 and
Ottawa Road 29, Arnprior, Ontario
MTO Agreement No. 4020-E-0012-32

1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., amalgamated with WSP in 2023) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a detailed foundation investigation associated with the proposed Arnprior Maintenance Patrol Yard (MPY) under Assignment No. 32 of the Eastern Region Retainer Mega 16 (Assignment No. 4019-E-0019).

In July 2022 WSP carried out a preliminary foundation investigation for the proposed Arnprior MPY development at the Highway 417 and Ottawa Road 29 (OR29) site. The results of the preliminary foundation investigation are contained in the following report:

- **MTO GEOCREs No. 31F-244:** “Foundation Investigation Report, Proposed Arnprior Maintenance Patrol Yard Location 12 - Highway 417 and Ottawa Road 29, Arnprior, Ontario” dated April 2023, prepared by Golder Associates Ltd.

This report presents the combined results of the preliminary and detailed design foundation investigation carried out for the preferred site of proposed for the MPY.

The scope of work for the foundation engineering services associated with the detail design and construction of the proposed MPY was outlined in Change in Scope - Sub-Consultant for Assignment 32 dated August 22, 2023. The detail design investigation program was developed to provide one (1) additional borehole for the revised Storm Water Management Pond and provide soil sampling for environmental testing for Dillon to develop an Excess Soil Management plan. The detailed design investigation report for the proposed MPY structures is based on the information gathered during the preliminary and detailed investigations.

All work has been carried out in accordance with WSP's Quality Control Plan for foundation engineering services for the project dated April 2021 provided to Dillon.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 Site Description

The proposed Arnprior MPY site is to be located between Upper Dwyer Hill Road and OR29, in the southeast quadrant of the Highway 417 / OR29 interchange southeast of Arnprior, Ontario. Site photographs showing the general conditions at the site during the field investigation in July 2022 and October 2023 are presented in Appendix D.

In the area of the MPY site, Highway 417 is divided with a four-lane cross-section with two eastbound and two westbound through lanes, plus speed change lanes at the interchange ramps. The existing underpass structure carries the two through lanes of OR29 over Highway 417. Upper Dwyer Hill Road is located to the south of the site and is a rural roadway with one through lane in each direction and gravel shoulders.

The proposed site is undeveloped, with generally flat to rolling topography and is covered with grass and bush with several low-lying areas.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*, the MPY site lies within the minor physiographic region known as the Ottawa Valley Clay Plain, which lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the former Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock¹. Bedrock in this region is within the geological boundaries between carbonate meta-sedimentary rocks consisting of marble of the Grenville Supergroup and Flinton Group, and limestone, dolostone and sandstone of the Ottawa Group and Simcoe Group.

3.0 INVESTIGATION PROCEDURES

The fieldwork for the preliminary investigation was carried out between June 27 and July 11, 2022, and included advancing eight boreholes, numbered 22-01 to 22-08 for the proposed buildings, and two boreholes, numbered SWMP1 and SWMP2 for the proposed Stormwater Management Pond (SWMP) at that time. Boreholes 22-01 to 22-04 were located at the four corners of the footprint for the proposed Vehicle Maintenance Garage (VMG), while boreholes 22-05 to 22-08 were located in the four corners of the footprint for the proposed Material Storage Building (MSB). The SWMP boreholes were located within the footprint of the then proposed stormwater management pond.

The fieldwork for the detailed design investigation, including the environmental sampling, was carried out between October 16 and 18, 2023. The investigation included advancing one borehole for the revised SWMP design (23-03) and ten boreholes for environmental sampling (23-201 to 23-210). The information gathered for the environmental sampling, including the field borehole logs, were provided to Dillon and are not discussed further in this report.

The locations of the proposed structures were provided by Dillon on November 29, 2023, and are shown in Drawings 1 to 3.

The preliminary and detailed design boreholes were advanced with a CME850 track-mounted drilling rig. The drilling equipment was supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. (CCC) of Ottawa, Ontario and Marathon Underground Constructors Corporation of Greely, Ontario, respectively.

Soil samples were obtained using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Soil samples were obtained at vertical sampling intervals of about 0.76 m. Relatively undisturbed samples of the clay were also retrieved from within the cohesive deposit using a fixed-piston sampler and thin-walled Shelby tubes.

In-situ vane testing was carried out within the cohesive deposits using an MTO N-size vane, with the reaction (torque) measured by a pair of calibrated scales, to measure undrained shear strengths. After measuring the undrained shear strength, remoulded shear strengths were also measured at selected intervals.

After sampling to a depth of approximately 3.0 m, Boreholes 22-02, 22-03, 22-05 and 22-08 were advanced to refusal without sampling using Dynamic Cone Penetration Testing (DCPT).

HQ- or NQ-sized bedrock core samples were obtained using the rotary diamond drilling technique and a triple-tube core-barrel at Boreholes 22-01, 22-04, 22-06, 22-07, and 23-03.

¹ Belanger, J.R. "Urban Geology of Canada's National Capital Area", in Urban Geology of Canadian Cities, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

A monitoring well was installed in Boreholes 22-01 and 23-03, to observe the stabilised groundwater level at the site. The monitoring wells consist of 32 mm outside diameter PVC tubing with 1.5 m long slotted screens installed within the silty clay (22-01) and till/bedrock (23-03). The groundwater levels were measured in the wells on July 15, 2022, October 4, 2022, and on December 12, 2023.

All boreholes without a monitoring well were backfilled with bentonite within the bedrock, and bentonite mixed with soil cuttings within the overburden. The boreholes containing monitoring wells were backfilled with bentonite mixed with cuttings to the underside of the well screen depth, followed by a sand pack to 0.6 m above the top of the well screen, followed by a minimum 1.5 m thick layer of bentonite to form a “cap” over the well screen, followed by a layer of bentonite mixed with cuttings to ground surface. All boreholes were backfilled in general accordance with the intent of O.Reg 903, as amended. The site conditions were restored following completion of the fieldwork.

The fieldwork was supervised on a full-time basis by members of WSP’s technical staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers, and transported to WSP’s laboratory in Ottawa for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses, and Atterberg limits testing were carried out on selected soil samples at WSP’s and Stantec’s Ottawa laboratories. Two undisturbed Shelby tube soil samples were submitted to Stantec’s Ottawa laboratory for one-dimensional consolidation testing of the sensitive silty clay. The uniaxial compressive strength (UCS) testing was carried out on selected samples of the bedrock at Stantec’s Ottawa laboratory. The laboratory tests were carried out to MTO LS and/or ASTM Standards, as appropriate.

One groundwater and two soil samples were sent to Eurofins Environmental Testing Canada Inc. (Eurofins) for basic chemical analysis related to potential corrosion of buried steel elements and sulfate attack on buried concrete elements (corrosion and sulphate attack).

The borehole locations and elevations were surveyed by WSP using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The borehole locations, including northing and easting coordinates, ground surface elevations, and drilled depths are summarized in Table 1.

Table 1: Summary of Borehole Locations

Borehole	NAD83 CSRS CBNv6-2010.0 MTM Zone 9		Ground Surface Elevation (m)	Drilled Depths (m)	Comments
	Northing (m)	Easting (m)			
22-01	5030649.6	318064.8	104.9	17.1	Bedrock Cored
22-02	5030669.0	318085.3	104.7	10.4 ¹	DCPT Refusal
22-03	5030628.4	318084.5	106.7	16.3 ¹	DCPT Refusal
22-04	5030646.3	318104.3	106.1	21.6	Bedrock Cored
22-05	5030718.7	318161.1	105.4	10.5 ¹	DCPT Refusal
22-06	5030751.7	318198.0	105.7	21.4	Bedrock Cored
22-07	5030694.9	318182.5	106.3	15.3	Bedrock Cored
22-08	5030734.8	318215.0	105.8	15.8 ¹	DCPT Refusal
SWMP1	5030544.4	317965.7	104.4	8.2 ²	-
SWMP2	5030562.4	317992.2	103.8	8.8 ²	-
23-03	5030591.3	317977.7	104.0	11.2	Bedrock Cored

Notes:¹ Borehole terminated at DCPT refusal² Borehole terminated within grey clay.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The subsurface soil, bedrock and groundwater conditions encountered in the boreholes and the results of in-situ testing from the investigation are shown on the Record of Borehole, and Drillhole sheets presented in Appendix A. The results of the laboratory testing carried out during the investigation are presented on the Record of Borehole sheets as well as in Figures B1 to B10 in Appendix B. The borehole locations and the interpreted stratigraphic profiles projected along the proposed MPY buildings and SWMP are provided in Drawings 1 to 3, respectively.

Photographs of the core samples recovered from the underlying bedrock are shown in Figures A1 to A10, provided in Appendix A. The results of the basic chemical testing/analysis completed on select groundwater and soil samples are provided in Appendix C.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections in Drawings 1 and 2, are inferred from observations of the drilling progress and noncontinuous sampling and therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

4.2 Site Stratigraphy Overview

At the boreholes, the subsurface conditions generally consist of topsoil and/or fill, overlying a very stiff weathered clay crust, transitioning to a firm to stiff grey silty clay to clay, underlain by bedrock. A layer of silty sand/sandy silt (till) to clayey silt-silt/silt (till) was encountered overlying the bedrock at some locations.

A more detailed description of the overburden soil deposits, and bedrock geology conditions encountered during the field investigation is provided in the following sections.

4.2.1 Surface Cover/ Surficial Materials

Topsoil with thickness ranging from 100 mm to 600 mm but more typically 200 mm was encountered at surface at all boreholes, except Borehole 22-06.

4.2.2 Fill

Fill consisting of silty sand to sandy silt to gravel and sand was encountered below the topsoil at Boreholes 22-03, 22-04, 22-07 and at ground surface at Borehole 22-06. The top of this layer was encountered at elevations ranging from 105.5 m to 106.6 m. The total thickness of this layer ranges from about 0.6 m to 2.0 m. The SPT N-values recorded in the fill range from 5 to 32 blows per 0.3 m of penetration but more typically 5 to 23 blows per 0.3 m indicating a loose to compact state of compactness. The measured moisture contents of four samples tested ranged from 5% to 29%. The results of grain size analysis testing carried out on four samples of fill material are provided in Figure B2 in Appendix B.

4.2.3 Clay to Silty Clay

A clay deposit was encountered beneath the surficial topsoil and or fill layers at all boreholes locations. The measured engineering properties of the clay deposit are shown on the plots in Figure B1 provided in Appendix B.

The upper portion of the clay deposit has a weathered, stiff to very stiff crust. The top of this layer was encountered at elevations ranging from 103.8 m to 105.6 m. The thickness of this layer at Boreholes 22-01, 22-04, 22-06, 22-07, SWMP 1, SWMP 2, and 23-03 where fully penetrated ranges from about 3.8 m to 5.3 m. The SPT N-values recorded in this layer range from 1 to 18 blows per 0.3 m of penetration but more typically 5 to 10 blows per 0.3 m. In-situ shear vane test results indicate the undrained shear strength of the grey-brown weathered clay ranges from 84 kPa to greater than 96 kPa, indicating a stiff to very stiff consistency. The ratio of the measured in-situ natural undrained shear strength to the remolded undrained shear strength ranges from about 5 to 10, as such the clay crust is classified as sensitive to extra sensitive in accordance with Section 3.1.3.4 the Canadian Foundation Engineering Manual (CFEM 2006).

The water content of fifteen samples of the clay crust ranges from 33% to 53%. The results of grain size analysis testing carried out on eleven samples of this material are illustrated in Figures B3 and B4 in Appendix B. The results of Atterberg limits testing completed on twelve samples of weathered clay crust indicate liquid limits ranging from 59 to 74, plastic limits ranging from 19 to 26 and plasticity indices ranging from 38 to 50. These Atterberg limits test results indicate a clay of high plasticity (CH). The Atterberg limits test results are illustrated in Figure B5 in Appendix B.

The silty clay to clay encountered below the weathered crust is grey. The top of this layer was encountered at elevations ranging from 99.2 m to 100.5 m. The thickness of this layer where fully penetrated ranges from about 5.0 m to 11.6 m. Boreholes SWMP1 and SWMP2 were terminated in this layer. The SPT N-values range from weight of rod (WR) to 4 blows per 0.3 m of penetration, but more typically WR to 1 blow per 0.3 m. In-situ shear vane test results measured the undrained shear strength of the grey silty clay to clay ranging from 38 kPa to 77 kPa, but more typically 38 kPa to 58 kPa, indicating a firm to stiff consistency. The ratio of the measured in-situ natural undrained shear strength to the remolded undrained shear strength ranges from 2 to 31, but more typically 5 to 13; indicating sensitivity to extra sensitive clay in accordance with Section 3.1.3.4 of CFEM (2006).

The water contents of the nine samples of the grey silty clay to clay range from 38% to 54%. The results of grain size analysis testing carried out on seven samples of this material are illustrated in Figure B6 in Appendix B. The results of Atterberg limits testing completed on eight samples of the grey clayey soils indicate liquid limits ranging from 37 to 57, plastic limits ranging from 17 to 22 and plasticity indices ranging from 18 to 36. These Atterberg limits test results indicate silty clay to clay of intermediate to high plasticity, but more typically intermediate plasticity silty clay (CI). The Atterberg limits test results are illustrated in Figure B7 in Appendix B.

Laboratory oedometer consolidation testing was carried out on two samples of the grey silty clay deposit. The preconsolidation pressures were estimated from the void ratio versus logarithmic stress plot (e - $\log \sigma'$) using the Casagrande method as well as using the work method (after Becker et al., 1987). The results of the testing are provided in the Consolidation Test Results report provided in Appendix B and are summarized in Table 2.

Table 2: Summary of Consolidation Testing

Borehole	Sample	Ground Surface Elevation (m)	Test Elevation (m)	Unit Weight (kN/m ³)	e_o	σ'_p (kPa)	σ'_{vo} (kPa)	$\sigma'_p - \sigma'_{vo}$ (kPa)	C_c	C_r	OCR
22-04	ST-11	106.1	95.1	16.9	1.406	155	90	65	0.719	0.016	1.7
22-07	ST-09	106.3	98.4	17.1	1.329	225	75	150	1.043	0.010	3.0

Notes:

σ'_p Estimated preconsolidation stress
 σ'_{vo} Estimated in-situ vertical effective stress
 C_c Compression index
 C_r Recompression index
 e_o Initial void ratio
 OCR Overconsolidation ratio

4.2.4 Clayey Silt

Clayey silt containing varying amounts of sand was encountered below the grey silty clay stratum at Borehole 22-01. The top of this layer was encountered at Elevation 93.6 m and the layer is 1.5 m thick. The measured SPT N-value within this layer is weight of hammer (WH). In-situ shear vane test results measured undrained shear strengths of 50 kPa and 58 kPa, indicating a stiff consistency within the clayey silt. The ratio of the measured in-situ undrained natural shear strength to the undrained remolded shear strength was 3 and 4, as such the clayey silt is classified as medium sensitive to sensitive in accordance with Section 3.1.3.4 of CFEM (2006).

The measured water content of a single sample of the clayey silt was 25%. The results of grain size analysis testing carried out on a single sample of this material is provided in Figure B8 in Appendix B. The results of Atterberg limits testing completed on a single sample of the clayey silt indicates a liquid limit of 25, a plastic limit of 14, and a plasticity index of 11. The Atterberg limits test results are shown in Figure B9 in Appendix B and indicate a clayey silt of low plasticity (CL).

4.2.5 Clayey Silt-Silt to Silt of Slight Plasticity (Till)

Clayey silt-silt to silt of slight plasticity (till) containing varying amounts of sand was encountered below the grey silty clay stratum at Borehole 23-03. The top of this layer was encountered at Elevation 95.0 m and the layer is 0.4 m thick. The recorded SPT N-value within this layer was 56 blows/0.25 m, suggesting a hard consistency, however the blow count has likely been influenced by the proximity to the bedrock surface rather than the actual consistency of the soil matrix.

The measured water content of a single sample from this layer was 12%. The results of Atterberg limits testing completed on a single sample of this material indicates a liquid limit of 16, a plastic limit of 11, and a plasticity index of 4. The Atterberg limits test results are shown in Figure B10 in Appendix B and indicate a clayey silt-silt to silt of slight plasticity (CL-ML/ML).

4.2.6 Silty Sand to Sandy Silt (Till)

A granular till layer was encountered below the clayey silt at Borehole 22-01, and below the silty clay at Boreholes 22-04 and 22-06. The till is described as consisting of silty sand to gravelly silty sand to gravelly sandy silt with trace to some clay and containing cobbles. Given the nature of tills in this area, it is anticipated that the glacial till layer likely also contains boulders. The top of this layer was encountered at Elevations 92.1 m and 90.4 m in Boreholes 22-01 and 22-04, respectively, and interpreted to be at Elevation 88.9 m in Borehole 22-06, and the layer is 0.4 m to 1.6 m thick. The recorded SPT N-values within this layer range from 50 blows/0.03 m to 50 blows/0.15 m, suggesting a very dense compactness, however the blow counts have likely been influenced by the presence of cobbles, boulders, or the proximity to the bedrock surface rather than the actual compactness of the soil matrix.

4.3 Bedrock/DCPT Refusal

The overburden materials are underlain by marble, dolostone, or sandstone bedrock with crystalline calcarenite limestone interbeds.

HQ or NQ-sized bedrock core samples were obtained using rotary diamond drilling technique and a triple-tube core-barrel at Boreholes 22-01, 22-04, 22-06, 22-07 and 23-03.

Table 3 summarizes the depths and the elevations of the bedrock surface as encountered at the borehole locations.

Table 3: Summary of Bedrock Surface Depths and Elevations

Borehole	Existing Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation ¹ (m)
22-01	104.9	13.2	91.7
22-04	106.1	16.8	89.3
22-06	105.7	18.4	87.3
22-07	106.3	11.8	94.5
23-03	104.0	9.4	94.6

Note(s):

1. Bedrock surface elevation confirmed by rock coring.

Marble bedrock was encountered below the till at Boreholes 22-01, 22-04 and 23-03. The top of the marble bedrock was encountered between Elevations 94.6 m and 89.3 m in the boreholes. Rock Quality Designation (RQD) values measured on the recovered marble bedrock core samples range from about 0% to 98%, but are typically 44% to 98%, indicating a poor to excellent rock quality. The result of uniaxial compressive strength (UCS) testing carried out on a single marble bedrock core sample gave a UCS value of 206 MPa, indicating a very strong bedrock.

Dolostone bedrock was encountered below the marble bedrock at Borehole 22-04, and below the grey silty clay in Borehole 22-07. The top of the dolostone bedrock was encountered at Elevations 86.3 m and 94.5 m in Boreholes 22-04 and 22-07, respectively. RQD values measured on the recovered dolostone bedrock core samples range from about 60% to 89%, indicating a fair to good rock quality. The result of UCS testing carried out on a single dolostone bedrock core sample gave an UCS value of 186 MPa, indicating a very strong bedrock.

Sandstone was encountered below the till at Borehole 22-06. Crystalline calcarenite-limestone interbeds are present in the bedrock core. The top of the sandstone bedrock was encountered at Elevation 87.3 m. RQD values measured for the sandstone bedrock range from about 39% to 77%, indicating a poor to good rock quality. The result of UCS testing carried out on a single sandstone bedrock core sample gave a UCS value of 26 MPa, indicating a medium strong bedrock.

The results of the laboratory UCS testing are attached in Appendix B.

Below a depth of approximately 3.0 m, Boreholes 22-02, 22-03, 22-05 and 22-08 were advanced to refusal without sampling, using Dynamic Cone Penetration Testing (DCPT). Table 4 summarizing the DCPT refusal elevations encountered during the investigation. DCPT refusal is defined where a blow-count of 100 blows/0.3 m (or greater) is measured in the test and is not necessarily indicative of top of bedrock surface since it is possible that the DCPT could reach refusal on cobbles and/or boulders within the till overlying the bedrock. For top of bedrock elevations, reference should be made to the bedrock elevations confirmed by coring as provided in Table 3.

Table 4: Summary of DCPT Refusal Elevations

Borehole	Existing Ground Surface Elevation (m)	DCPT Refusal Elevation (m)
22-02	104.7	94.3
22-03	106.7	90.4
22-05	105.4	94.9
22-08	105.8	90.0

4.4 Groundwater Condition

Monitoring wells were installed in Boreholes 22-01 and 23-03 to measure the groundwater level at the site. Table 5 summarizes the groundwater levels measured in the monitoring wells.

It is expected that the groundwater levels will be subject to fluctuations both seasonally and as a result of precipitation events.

Table 5: Summary of Groundwater Conditions

Borehole	Screened Interval	Ground Surface Elevation (m)	Ground Water Depth (m)	Ground Water Elevation (m)	Date
22-01	Grey Silty Clay	104.9	1.3	103.6	July 15, 2022
			0.9	104.0	October 4, 2022
			2.4	102.5	December 12, 2023
23-03	Till / Bedrock	104.0	0.8	103.2	October 19, 2023
			5.2	98.8	December 12, 2023

4.5 Steel Corrosion and Sulphate Attack, Chemical Analysis

One groundwater sample and two soil samples were submitted to Eurofins for chemical testing/analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix C and are summarized in Table 6 and Table 7.

Table 6: Steel Corrosion and Sulphate Attack, Chemical Analysis – Water Sample

Borehole	Chloride (mg/L)	Sulphate (mg/L)	Electrical Conductivity (mS/cm)	pH	Resistivity (Mohm-cm)
22-01	13	62	425	8.26	<0.2

Table 7: Steel Corrosion and Sulphate Attack, Chemical Analysis – Soil Samples

Borehole	Sample Depth (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
22-06	3.1-3.7	0.003	0.06	0.13	8.29	7,690
22-07	4.6-5.2	0.003	0.04	0.17	8.14	5,880

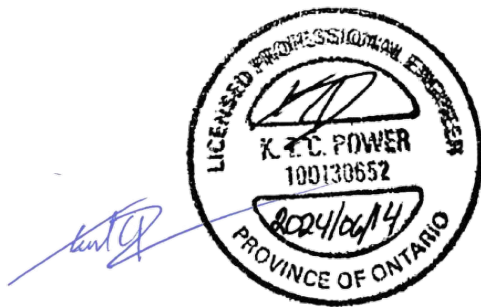
5.0 CLOSURE

This report was prepared by Kinjal Gajjar a Geotechnical Consultant with WSP and Kenton Power, P.Eng. The report was reviewed by Paul Dittrich, P.Eng. a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP conducted an independent technical and quality review of this report.

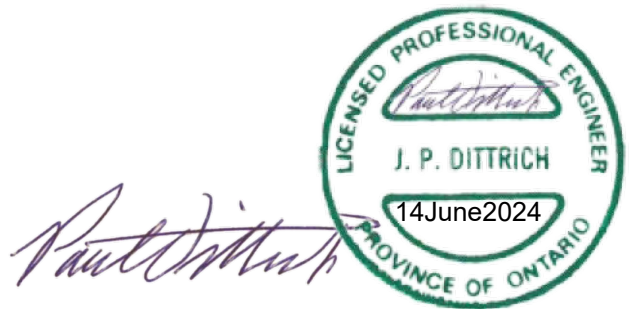
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PART B

Foundation Design Report

Detailed Design Proposed
Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 and
Ottawa Road 29, Arnprior, Ontario
MTO Agreement No. 4020-E-0012-32

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical and foundation design input associated with the detailed design of the proposed Arnprior Maintenance Patrol Yard (MPY) as part of the Assignment No. 32 of the Eastern Region Retainer Mega 16 (Assignment No. 4019-E-0019). The input provided herein is based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigations, and in accordance with the procedures outlined in NBCC2015 for Limit States Design.

The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.1 Proposed Works

The foundation design recommendations provided herein are based on the proposed construction at the Arnprior MPY site as detailed on *Sheet 13 New Construction MPY Site* drawing from the 60% drawing package for this project dated November 23, 2023, provided by Dillon. It is our understanding that the proposed MPY is to consist of a material storage building (MSB) where the road salt and sand is to be stored, a vehicle maintenance and administration building (VMB) and other small outbuildings, at grade asphalt parking lots and roadways and a storm water management pond (SWMP). Drawings 1 to 3 show the proposed layout of the main components for the MPY facility.

6.1.1 Grade Raise

A grade raise ranging from 0.5 m in height (to the south) to 2.5 m in height (to the north) is required at the site as part of proposed construction. The grade raises fill thicknesses used in the analyses are based on the contour cut/fill plans provided by Dillon on November 29, 2023. Drawing 3 illustrates the proposed cut/fill contours across the site. The proposed grade raise over the large area of the MPY site will result in time-dependent settlement of the clayey foundation soils and as such, settlement mitigation measures will need to be considered as part of the overall design.

6.1.2 Material Storage Building (MSB)

The dimensions of the MSB are indicated on *Sheet 108 Material Storage Building – Foundation Plan, Footing/Pier* from the 60% drawing submission package dated November 23, 2023, provided by Dillon. The MSB is to have an overall approximate footprint of 36.2 m x 48.9 m. The footprint area where the salt material will be stored is approximately 20.7 m x 30.0 m. The proposed clear internal height at the centre of the building is approximately 12.5 m. To maximize storage capacity, no internal columns are proposed. The building is to be supported on 3.5 m wide perimeter foundations. As the site soils are frost susceptible and the MSB is to be unheated, footings/pile caps will need to be founded at a depth of 1.8 m below final finished grade or be suitably insulated. It is understood that the proposed construction for the MSB is to include an asphalt floor.

It is our understanding that the design of the MSB also includes 3 m high push walls (i.e., foundation walls) along four sides of the structure which are to be supported on the perimeter foundations. As outlined in the Preliminary Investigation report (MTO GEOCRETS No. 31F-244) due to the compressive nature of the clayey soils, the push walls at the MSB are to be supported on deep foundations driven to bedrock.

The salt pile within the MSB will be 3 m tall at the push wall and then continue upwards in a pyramidal shape at the angle of repose of the salt (assumed to be 32°) to a maximum height estimated to be approximately 9.4 m. The salt pile is to taper down to ground surface/floor level elevation at the open front of the MSB. These dimensions/geometries have been used to evaluate the loading from the salt pile on the underlying foundation soils.

6.1.3 Vehicle Maintenance Building (VMB)

Footing sizes and layout for the VMB are based on *Sheet 101 – Vehicle Maintenance Building - Foundation Plan, Footing/Pier Schedule* from the 60% drawing package dated November 23, 2023, provided by Dillon. The VMB is to have an overall footprint of 32.9 m x 30.1 m. Table 8 outlines the various proposed footings sizes at the VMB.

Table 8: Vehicle Maintenance Building Footing Schedule

Building	Grid Line	Location	Size (m)
Vehicle Maintenance	A – A.1 and G – G.1	External Columns 4 to 6	3.0 x 3.0
	8	External Columns B to F	2.5 x 2.5
	2	Internal Columns B to F	2.5 x 2.5
	-	External Corner footings	6.0 x 3.0
Administration	1	External Columns A to G	1.8 x 1.8

6.2 Seismic Design

6.2.1 Seismic Hazard and Importance Category

The seismic hazard values associated with the design earthquake are those established for the National Building Code of Canada (NBC 2020) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 6th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2020. As described in Section 6.3.2 frost penetration depth at this location is 1.8 m. The following sections assume the founding elevations for the proposed exterior footings or pile caps will be at the frost penetration depth of 1.8 m below ground surface.

6.2.2 Seismic Site Classification

In accordance with Table 4.1.8.4.A of the NBCC 2015, the selection of the seismic site classification is based on the soil and bedrock conditions encountered in the upper 30 m of the stratigraphy below the founding elevation.

Based on the current understanding of the foundation conditions at the site, the site would be classified as a Seismic Site Class D.

6.2.3 Spectral Response Values

In accordance with Section 4.1.8.4.1 of the NBC2015 and based on the location of the proposed Arnprior MPY, the Class D peak seismic hazard values for the site based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca) are provided in Table 9.

Table 9: Site Class D Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year) (g)
PGA	0.328
$T \leq 0.2$ s	0.553
$T = 0.5$ s	0.478
$T = 1.0$ s	0.286
$T = 2.0$ s	0.137
$T = 5.0$ s	0.0382
$T \geq 10.0$ s	0.012

6.3 MSB and VMB Building Foundations

6.3.1 Subsurface Conditions

The subsurface conditions at the site generally consist of topsoil and fill, overlying a weathered clay crust, over grey clay to silty clay, over glacial till, underlain by bedrock.

In general, the weathered clay crust has a stiff to very stiff consistency and can provide a moderate geotechnical (bearing) resistance for shallow footings of conventional width. The clay to silty clay below the weathered crust has a generally firm to stiff consistency and would provide a low (geotechnical) bearing resistance for shallow footings and be susceptible to excessive settlement under even moderate loads. The bedrock surface is between about 12 m and 18 m below the existing grade and suitable for founding driven piles.

6.3.2 Frost Protection

As per Ontario Provincial Standard Drawing (OPSD) 3090.101, the frost penetration depth at the site is estimated to be 1.8 m below ground surface. Footings constructed at this site, or the underside of pile caps, should have a minimum embedment depth of 1.8 m below final finished grade for frost protection purposes.

Floor slabs in unheated buildings should be provided with frost protection in the form of rigid extruded polystyrene insulation installed beneath the slab. For design of the insulation, it can be assumed that a 25 mm thickness of insulation will provide the equivalent frost protection of 0.3 m of conventional soil cover. As such, for the design frost penetration depth at this site (i.e., 1.8 m deep), 150 mm of rigid insulation will be required below the floor slab. Depending on the design loads on the floor slab, a high compressive strength insulation may be required.

6.3.3 Foundation Design Alternatives

The weathered clay crust deposit has the strength and stiffness to support conventional, nominally loaded shallow foundations and it is anticipated that the footing loads at the vehicle maintenance building and other lightly loaded structures at the site can be supported on shallow foundations founded within this stratum. The underside of any shallow footings should be founded no lower than approximately Elevation 103 m to ensure that there is sufficient

clay crust thickness beneath the footing(s) to provide adequate bearing resistance while still satisfying the minimum protection/cover requirements for frost protection.

At the MSB, given the high vertical and lateral loads imposed by the salt pile, shallow foundations are not considered a feasible foundation option as the native firm clayey strata below the crust will not provide sufficient bearing capacity/lateral resistance to support perimeter/push wall foundation loadings without excessive settlement. As such, deep foundations founded on or in the bedrock are required to support the MSB perimeter foundations. After incorporating the minimum frost cover requirements for pile caps and considering the approximate grade raise thickness of 2.0 m across the footprint of the MSB, it is estimated that the piles would range in length from about 11 m to 18 m. As part of the assessment of deep foundation alternatives for this site, both driven steel H-piles, and concrete caissons (drilled shaft piles) have been considered; however, from a foundation's perspective, the use of a driven steel H-pile is considered the preferred alternative to support the perimeter foundations for the material storage building.

A comparison of the foundation alternatives (deep and shallow), including advantages, disadvantages, risks and relative costs is provided in Table 18 following the text of this report.

6.3.4 Recommended Foundation Types

Based on the evaluation of foundation alternatives discussed above, the preferred approach from a foundations perspective is to support the Material Storage Building (MSB) founding elements on steel H-piles driven to bedrock, and the remaining structures including the VMB on shallow foundations within the grade raise material or undisturbed very stiff native weathered clay crust.

6.4 Foundation Recommendations

6.4.1 Shallow foundations

Shallow spread footings up to a maximum width of 3.0 m and founded within either the undisturbed native, weathered very stiff clay crust or within the compacted granular engineered fill for the site grade raise, at 1.8 m (frost depth) below finished grade, but not lower than Elevation 103 m, may be designed based on the following geotechnical resistances.

- Factored ultimate geotechnical resistance: 175 kPa
- Factored serviceability geotechnical resistance (for up to 25 mm of settlement) – not considering effects of grade raise: 150 kPa
- Factored serviceability geotechnical resistance (for up to 25 mm of settlement) – considering effects of the grade raise a seven-month preload with a 1 m high surcharge and a maximum footing stress of 100 kPa

As illustrated on Figures F6 and F7 in Appendix F to limit the maximum post-construction settlement of the VMB footings to 25 mm, the VMB may be designed with a factored serviceability geotechnical resistance of 100 kPa. It is noted that this presumes that the footings sizes do not change from those shown in the 60% Design Submission Drawings and summarized in Table 8.

6.4.1.1 Sliding Resistance

Resistance to lateral forces through sliding resistance between the underside of the concrete footing(s) and the underlying soils should be evaluated using the unfactored coefficients of friction provided in Table 10.

Table 10: Unfactored Coefficients of Friction between Footing and Founding Material

Footing Material	Founding Material	
	Native Clay	Granular A
Cast-in-place concrete	0.35	0.55
Precast concrete	0.30	0.40

6.4.2 Deep Foundations

The following two standard steel H-piles sizes have been used in the deep foundation assessment:

- 310 x 110 steel H-pile
- 310 x 132 steel H-pile

The perimeter/push walls of the material storage building may be founded on either pile size noted above driven to refusal in/on sound bedrock. The estimated pile lengths required to support the MSB foundations, assuming that the underside of the pile caps is founded 1.8 m below final ground surface and the piles are driven to bedrock, are summarized in Table 11.

Table 11: Estimated Pile Tip Elevations Material Storage Building

Reference Borehole	Approximate Underside of Pile Cap Elevation ¹ (m)	Estimated Pile Tip Elevation ² (m)	Estimated Pile Length (m)
22-06	105.2	87.3	18
22-07		94.5	11

Note:

¹ Underside of pile cap assumed based frost penetration depth after the construction of 2.0 m thick grade raise across the building footprint.

² Bedrock surface elevation at the borehole location

Piles must be installed in accordance with OPSS.PROV 903 Section 903.07.02.07.03.03, Piles to Bedrock.

6.4.2.1 Factored Geotechnical Axial Resistance

Based on the estimated uniaxial compressive strength of the bedrock at this site and assuming good to excellent rock quality, the following factored axial geotechnical resistance(s) can be used for design:

- 310x110 steel H-pile - factored geotechnical resistance at ULS of 2,600 kN per pile.
- 310x132 steel H-pile - factored geotechnical resistance at ULS of 2,700 kN per pile.

The factored ULS geotechnical resistance may be greater than the structural capacity of the pile, which could govern design and should be checked by the structural design engineer.

SLS does not apply to piles founded on or in the type of bedrock at this site, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Based on the bedrock surface encountered at the boreholes across the footprint of the material storage building, sloping bedrock should be anticipated. In addition, cobbles and boulders could be encountered within the till above the bedrock. As such, all pile tips will require protection from damage during driving and consideration must also be given to potential difficulties seating the piles during driving due to the presence of the sloping bedrock. As a result, it is recommended that piles be fitted with appropriate driving shoes and rock point as per OPSS.PROV 903 Section 903.07.02.02 (Driving Shoes and Rock Points); the APF Hard-Bite points (Model: HP-77750-B) or equivalent are recommended for this site. Non-Standard Special Provisions (NSSPs) should be included in the Contract Documents to warn the Contractor of the potential for the presence of obstructions (cobbles and boulders) in the overburden as well as the sloping bedrock. Example text for these NSSPs has been provided in Appendix G.

6.4.2.2 *Downdrag*

The piles installed at the material storage building will be subjected to downdrag loads due the grade raise at the site as well as the loading from the salt pile. Any unsupported areas subject to loading will experience consolidation settlement of the underlying clay to silty clay strata due to these loads. Settlement and downward movement of the cohesive deposit relative to the stiff end-bearing piles will result in the development of downdrag loads (negative skin friction) on the piles, especially if the piles are installed prior to completion of the settlement.

The structural design of the piles should be based on the full downdrag load acting on the piles. The estimated unfactored downdrag load acting on a single HP310X110 pile or HP310x132 pile is 425 kN for the conditions at the MSB. Downdrag loads could be reduced if construction is staged to include a sufficiently long preloading/surcharging period to complete the consolidation settlements prior to pile installation. However, if the construction schedule constraints will not allow the settlements to be completed prior to pile installation, the full downdrag loads must be considered in the structural design of the piles.

The downdrag loads noted above are unfactored loads. The structural capacity of the piles must be checked for the factored dead and downdrag loads.

6.4.2.3 *Resistance to Lateral Loads on Piles*

Resistance to lateral loads acting on the perimeter/push wall foundations of the MSB may be derived using vertical piles, with enhanced support offered by inclined (battered) piles, if required. For vertical piles, the resistance to lateral loading will be derived solely from the soil in front of the piles, whereas inclined piles derive lateral resistance from the soil in front of the pile as well as from the horizontal component of the axial load present in the inclined pile.

For a single driven HP 310x110 or HP 310x132 pile, the estimated factored lateral resistance at ULS is 150 kN. The estimated serviceability geotechnical resistance for 10 mm of horizontal deflection at the pile cap is estimated to be 110 kN.

Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a pile to lateral loads is highly non-linear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behaviour of the soil should be considered using P-y curves.

The lateral response of the vertical piles under lateral loading at this site may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equation given below, as described by Terzaghi (1955) in CFEM (1992).

$$k_h = \frac{67s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,
 B is the pile diameter/width (m).

Table 12 provides the recommended range for the value of $S_{u\text{mob}}$ to be used in the lateral structural analysis. The range in values reflects the variability in the subsurface conditions and values used will depend on the design elevation of the pile cap. Design values are provided for the full stratigraphic sequence at the site. The design shear strength profile is also shown in Figure B provided in Appendix E.

Table 12: Geotechnical Design Parameters for Lateral Resistance of Piles

Cohesive Stratum	Elevation (m)	Corrected Undrained Shear Strength, $S_{u\text{mob}}$ (kPa)
Cohesive Stratum	105 to 99.5	75 to 50 decreasing with depth
Grey Clay (CH)	99.5 to 96	50 to 40 decreasing with depth
Grey Silty Clay (CI)	96 to 89	40 to 50 increasing with depth

Group action for lateral loading should be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the Commentary to the CHBDC (2019).

6.5 Lateral Earth Pressures for Design for Retaining/Perimeter Walls

6.5.1 General

The perimeter/push walls planned for the material storage building will serve as retaining walls supporting the base of the salt stockpile. The lateral earth pressures acting on retaining walls depends on the type and method of placement of the backfill materials, the nature of the soils behind the backfill (if any), the magnitude of any surcharge loading including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. For the design of the push walls for the MSB, the salt will act as the backfill behind the wall.

The lateral earth pressure parameters provided below assumes that the salt 'backfill' material is fully drained so that there are no unbalanced hydrostatic pressures acting on the wall. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures behind the retaining walls should be considered in the design.

Based on the information provided by Dillon, it is our understanding that the salt stockpile will be placed in the material storage building to a height of 3 m at the push wall and will slope upwards away from the top of the wall at the natural angle of repose of salt of 32° . It is assumed the push walls will be unrestrained and, in this case, active lateral earth pressures should be used for the design. If lateral movement is not permissible and/or if the wall is retained from lateral yielding, it is recommended that at-rest horizontal lateral earth pressures be used for design.

6.5.2 Lateral Earth Pressures Coefficients

Table 13 provides the unfactored lateral earth pressure parameters to be used for design of the perimeter/push walls retaining the salt stockpile having a backslope of 32° .

Table 13: Static Lateral Earth Pressure Coefficients

Soil Type	Internal Angle of Friction (ϕ°)	Unit Weight (γ , kN/m ³)	Coefficients of Earth Pressure	
			At-Rest, K_o	Active, K_a
Salt	32	13	0.72	0.70

6.6 Assessment of Global Stability of Salt Pile

6.6.1 General

The global stability of the salt storage pile was evaluated using limit equilibrium analysis and the GeoStudio 2023.1.0 Slope/W software. The geometry used in the stability analysis was based on the topographic survey for the site, including the proposed grade raise, as well as the information on the proposed stockpile geometry at the MSB as outlined in Section 6.1.

For the stability analyses, and in the context of the CHBDC (2019), the target FoS is defined as being equal to the inverse of the product of the consequence factor, Ψ and the geotechnical resistance factor, ϕ_{gu} , (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, for a 'typical' consequence level and a 'typical' degree of site and prediction model understanding, a target minimum FoS of 1.33 and 1.54 has been used for the design of the berm slopes, considering global stability for temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019).

The full height of the salt pile was analysed under undrained (short-term), drained (long-term), and seismic design conditions.

The following geometry/parameters were used in the analysis:

- An average grade raises across the footprint of the MSB of 2.0 m with an assumed unit weight of 22 kN/m³;
- Push walls will be constructed along three sides of the salt pile supported on the perimeter footings founded on piles driven to bedrock;
- Footprint area where the salt material will be stored is approximately 20.7 m x 30.0 m;
- The applied stress of the salt pile was calculated based on the height and footprint of the salt pile (Section 6.1) and an estimated unit weight of road salt of 13 kN/m³;

- The salt pile will be 3 m tall at the push wall and then continues upwards in a pyramidal shape at the angle of repose of the salt (assumed to be 32°);
- A seismic horizontal loading of 0.164g, equal to ½ of the site adjusted PGA value (0.328g Site Class D) was used for seismic analysis (see Section 6.2);
- Soil stratigraphy was based on Section B-B shown in Drawing 1; and,
- Groundwater level at Elevation 105.5 m, assumed at the base of the grade raise.

6.6.2 Corrected Undrained Shear Strengths

As part of the assessment of the operative undrained shear strength(s) to be used in the short-term, total stress stability analysis and in the pseudo-static stability analysis, the field vane measured undrained shear strengths were adjusted following the procedure recommended in ASTM D2573 and using the correction factor proposed by Bjerrum et. al. (1972), using the following equation where μ is a function of plasticity index:

$$S_{u(mob)} = \mu \cdot S_{u(FV)}$$

As shown in Figure E-B in Appendix E, there is a variation in the plasticity index values (i.e., decreasing with depth) throughout the clay stratigraphy at the site. The shear strength correction factor(s), μ , were therefore selected based on the results of the index testing carried out for the current investigation following Bjerrum et al. (1972) with the results shown in Figure E-B.

6.6.3 Stability Analysis

The geotechnical design parameters used for the short-term/total stress stability analysis and pseudo-static stability analyses (i.e., undrained shear strengths, s_u) and for the long-term/effective stress stability analysis (i.e., drained shear strengths, c' and ϕ') are summarized in Table 14. These values have been selected for design based on the in-situ testing (i.e., field vane tests and SPT tests) in the boreholes and the results of the laboratory testing from the current investigation and the values adjusted based on precedent experience in similar soils.

The design $s_{u(mob)}$ profile used in the stability analysis is shown in Figure E1 in Appendix E.

The drained shear strength parameters (i.e., the effective angle of friction, ϕ' and effective cohesion, c') provided in Table 14 for the clay strata are based on the parameter assessment for strength and slope stability in Champlain Sea clays presented in the paper by Lefebvre (1981).

Table 14: Geotechnical Design Parameters for Stability Analysis

Material	Elevation (m)	Bulk Unit Weight, γ (kN/m ³)	Adjusted Undrained Shear Strength, $s_{u(mob)}$ (kPa)	Drained Analysis	
				Effective Angle of Friction, ϕ' (°)	Effective Cohesion, c' , (kPa)
Salt	116.9 (maximum)	13	-	32	-
Granular Fill (grade raise)	107.5 to 105	22	-	35	-
Weathered Crust (CH)	105 to 99.5	18	75 to 50 decreasing with depth	35	5
Grey Clay (CH)	99.5 to 96	17	50 to 40 decreasing with depth	35	5
Grey Clay (CI)	96 to 89	18	40 to 50 increasing with depth	30	5
Silty Sand (SM)	89 to 87	20	-	32	-

The results of the short-term/total stress and the long-term/effective stress stability analysis indicate that the salt pile has a factor of safety (FoS) greater than 1.5 for a deep-seated slip surface that could affect the stability of the perimeter walls for the MSB. Under the design earthquake loading, the salt pile has a FoS greater than 1.1. The results of the slope stability analysis are provided in Figures E2 to E4 in Appendix E.

6.7 Settlement Analyses at VMB and MSB

6.7.1 General

Settlement analyses were carried out using Settle3 (v5.013) by Rocscience to estimate the magnitude of settlement of the foundation soils at the VMB and MSB due the loading from the grade raise across the site, the shallow foundations at the VMB, and the salt and sand stockpiles at the MSB. As discussed in Section 6.3.4, it is recommended that the perimeter foundation walls of the MSB be supported on steel H-piles driven to bedrock and it is our understanding that this recommendation has been adopted in Dillon's detail design. As such, the loading from the perimeter foundations walls at the MSB has not been considered in the settlement analysis.

The settlement analyses were carried out assuming the following:

- Design life of the structures of both 50 years and 75 years;
- A maximum allowable settlement post-construction settlement at the shallow foundations of 25 mm;
- A variable thickness grade raises across the site (and building footprints) as illustrated on Drawing 3;
- The applied stress at the base of the salt pile calculated based on the height and plan dimensions of the salt pile (Section 6.1) and an estimated unit weight of road salt of 13 kN/m³;

- The settlement analysis conservatively assumes that the salt pile is maintained at its full height following completion of construction for the design life of the structure.
- Design footing load on the shallow foundations at the VMB building of both 150 kPa and 100 kPa at SLS; and,
- The consolidation parameters for the clayey strata are based on the results of the in-situ and laboratory testing completed during the previous and current investigations, as summarized in Figure F1 in Appendix F.

6.7.2 Preconsolidation Pressure (σ'_p) Values

As outlined in Section 4.2.3, incremental loading oedometer consolidation testing was carried out on two specimens of the unweathered clay sampled during the current investigation. The clay can generally be characterized as intermediate to high plasticity and is sensitive to extra sensitivity as indicated by the Atterberg limits test results and illustrated in Figure B1 in Appendix B.

In addition to the values of preconsolidation pressure estimated from the laboratory consolidation tests, preconsolidation pressures were also estimated for the clayey strata following the procedure outlined in Mesri et al. (1975). This procedure correlates the mobilized undrained shear strength ($s_{u(mob)}$) with the preconsolidation pressure (σ'_p) based on the measured plasticity index (I_p) of the soil strata, using the adjustment proposed by Bjerrum et al. (1972) as described in Section 6.6.2, and in accordance with the following equation:

$$\sigma'_p = s_{u(mob)}/0.22$$

The resulting values estimated for the preconsolidation pressure (σ'_p) along with the design line profile used in the analyses are shown in Figure F1 provide in Appendix F.

6.7.3 Settlement Analyses

The geotechnical design parameters used for the settlement analysis, based on the results of the previous and current investigations, are summarized below in Table 15 and plotted in Figure F1 in Appendix F. A schematic drawing of the loading conditions assumed in the settlement analyses for the two buildings are shown in Figures F2 to F11 in Appendix F.

Table 15: Geotechnical Design Parameters for Settlement Analysis

Material	Bulk Unit Weight (kN/m ³)	Elevation (m)	Preconsolidation Pressure σ'_p (kPa)	e_o	C_c	C_r	C_v (n/c) (10 ⁻⁴ cm ² /s)	C_{vr} (o/c) (10 ⁻³ cm ² /s)	C_{α}/C_c	E_s (MPa)
Weathered Crust (CH)	18.0	105.5	-	-	-	-	-	-	-	30 ¹
Grey Clay (CH)	17.0	99.5	250-150 Decreasing with depth	1.4	1.1 – 0.7	0.032 – 0.015	2.8	3.3	0.06	-
Grey Silty Clay (CI)	18.0	96.0	150-250 Increasing with depth	1.4– 1.0	0.7	0.015	2.8	2.2	0.06	-
Silty Sand (SM)	20.0	89.0	-	-	-	-	-	-	-	10

Note(s): 1. The weathered crust at the site has been modelled as an elastic material with $E'=30$ MPa based on precedent experience from other projects.

6.7.4 Results (Unmitigated)

The results of the settlement analysis for the scenario where the site grade raise and buildings are constructed sequentially (i.e., without any delay period) and assuming a load on the VMB footings of 150 kPa, are shown in Figures F2 and F3 for the VMB, and in Figures F4 to F6 for the MSB and are summarized below in Table 16.

Table 16: Initial Total Settlement Estimates

Structure	Design Life (years)	Estimated Maximum Total Settlement ² (mm)
Vehicle Maintenance Building	50	60
	75	65
Material Storage Building	50 ¹	115
	75 ¹	130

Note(s):

1. Assumes that the salt pile is maintained at full height for the design life under consideration.
2. Total Settlement includes immediate settlement, consolidation settlement and secondary compression (creep).

Based on the settlement analysis, the estimated total long-term settlements are greater than the typical structural design tolerance of 25 mm and as such, some form of settlement mitigation measures are required.

6.8 Settlement Mitigation Measures

The following sections outline mitigation measures that can be considered to manage or reduce the post-construction settlements associated with the site grade raise and construction of the proposed Arnprior MPY buildings. A summary of each mitigation option is presented, and recommended mitigation measure(s) are provided. The advantages, disadvantages, risks/consequences and relative costs for each mitigation option are summarized and ranked in Table 19 and Table 20 provided at the end of this report.

Due to the thickness of the compressible clayey soil strata (i.e., up to 15 m thick), and the presence of the stiff to very stiff weathered crust within the upper portion of the foundation stratum which will help support the new loads, full or partial sub-excavation of the cohesive deposit is not considered to be a practical option and is not discussed further herein.

6.8.1 Preloading plus Surcharging without Wick Drains

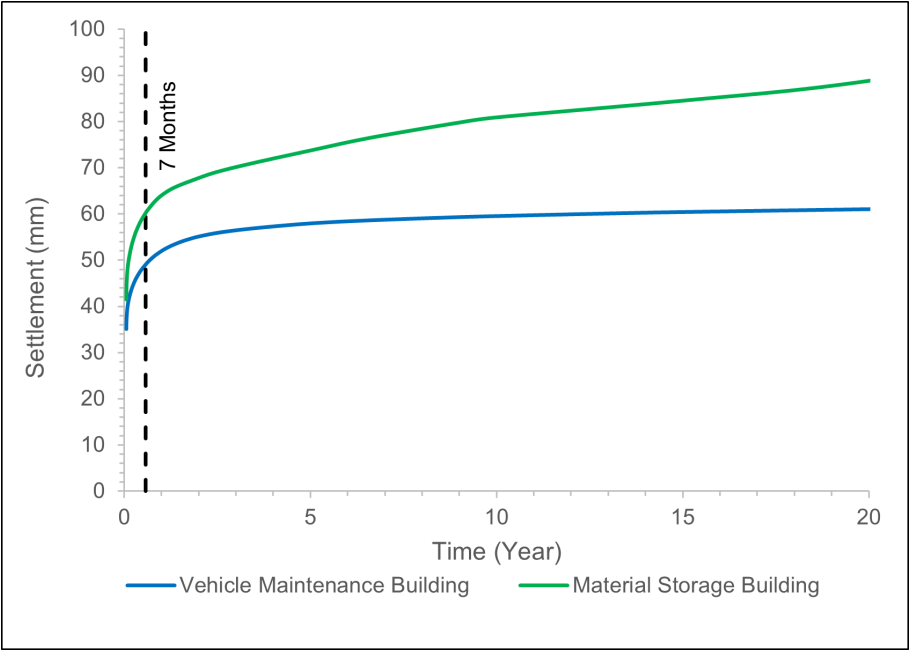
Preloading to compress the foundation soils and reduce the post-construction settlements of the proposed buildings is a feasible option for this site. Deep foundations would still be required for support of the MSB building foundations unless very high preload/surcharge load was incorporated across the MSB building footprint which could result in stability concerns.

Considering that a site-wide grade raise is required at the site as part of the development, if the construction schedule permits, the grade raise could be placed at the site ahead of constructing the buildings to act as a preload for a period to allow the site foundation soils to consolidate and settle such that the remaining post-construction settlements could be tolerated by the design or managed through maintenance.

Preloading requires placement of fills and monitoring of settlements, and possibly pore pressures, until such time that sufficient settlements have occurred to allow the next phase of construction to proceed. The preload time period is a function of the properties and thicknesses of the cohesive deposits as well as the anticipated loading stress and the magnitude of the tolerable post-construction settlement.

Preload times can be reduced by incorporating a temporary surcharge fill on top of the preload fill. The surcharge fill would apply a greater stress to the underlying cohesive foundation soils and increase the rate of settlement over that achieved by preloading alone. Given material handling costs and the proposed construction schedule, a 1.0 m thick surcharge above the proposed site grade raise at the VMB is considered appropriate at this site. The surcharge load would need to be constructed over the entire footprint of the VMB and 3 m beyond the perimeter of the building in all directions and then tapering down with 1H:1V side slopes.

Based on discussions with Dillon and the MTO, it is our understanding that a 7-month preload/surcharge period can be accommodated in the construction schedule. The figure below Graph 1 summarizes the estimated settlement versus time based on: (i) constructing the site grade raise and a 1 m surcharge prior to construction of the VMB; and, (ii) constructing the site grade raise only as a preload at the MSB. A surcharge load is not considered to provide appreciable benefit at the MSB as the total settlements are much larger than at the VMB and the relatively short preload/surcharge period that can be managed in the construction schedule has only a marginal effect on reducing the total post-construction settlements in this area; an additional mitigation measure (i.e., post-construction maintenance in the form of asphalt padding) will be required at the MSB.



Graph 1: Time vs. Estimated Settlement Preloading Mitigation

As shown in the Graph 1 at the VMB, a relatively large portion (greater than 70%) of the estimated total settlements listed in Table 16 occurs within the first seven months after construction of the grade raise. At the MSB, a smaller portion (less than 50%) of the estimated total settlement listed in Table 16 occurs within the first seven months, and therefore some additional mitigation measures (discussed in Section 6.8.4 below) will be required.

Wick drains can be used to accelerate consolidation in softer clays but are not recommended at this site given the over-consolidated nature of the clay stratum and considering that the final effective stresses due to the new construction (and salt stockpiling) will generally not exceed the preconsolidation stress. Although wick drains installed prior to placement of the preload fills would accelerate the consolidation process (i.e., even in the recompression range) during preloading, the presence of the wick drains would also accelerate the swelling/rebound of the clays during removal of any surcharge which is required at this site prior to construction of the VMB and would also occur during use (i.e., removal and re-filling of the salt pile) at the MSB. Without wick drains, the time for swelling/rebound would be much slower and as such, the magnitude of swelling/rebound would be less significant and therefore the preloading more effective in reducing the post-construction settlements at this site. Further, given the very stiff and thick, over-consolidated crust at this site, wick drain installation may be difficult and require pre-drilling which would add to the overall costs for this option.

The disadvantages of this mitigation option are the additional time added to the construction schedule, and the extra material handling cost of the 1 m thick surcharge fill material placed at the VMB. However, it is our understanding that the 7-month period can be accommodated in the construction schedule, and the 1 m of temporary surcharge fill can be re-used in other areas of the site.

6.8.1.1 Instrumentation and Monitoring

For the preloading/surcharging mitigation option, it is recommended that the magnitude, time rate of settlement, and the dissipation of pore pressures during and after construction of the preloading/surcharge fill pile be assessed with monitoring instrumentation.

Monitoring could consist of installing settlement plates/rods (SPs/SRs), surface settlement point (SSPs), or potentially vibrating wire in-line extensometers (VWIXs), as well as vibrating wire piezometers (VWPs) below the preloading/surcharge fills and taking regular measurements/readings at given intervals of time during and after fill placement. An additional VWP should also be installed outside the zone of influence of the preloading/surcharge fill pile to provide background pore pressure readings to properly assess the excess pore pressures. Careful consideration will have to be given to the location of the instruments to avoid damage during construction.

If a monitoring program were to be implemented to evaluate the long-term, post-construction performance of the pavement structure below the salt storage area, instrumentation should be selected that will minimize interference with the operation of the facility. In this scenario, the use of settlement plates/rods (SRs) should be avoided; instead, a non-intrusive monitoring program consisting of either simple surface settlement points (SSPs) such as nail pins (PK nails) embedded in the asphalt, or a reflectorless monitoring system, should be considered. Alternatively, a more elaborate VWIX system that could be fully buried below the pavement structure could be considered, but the additional costs for such a system are likely not justifiable in this application.

Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring would need to be included as an NSSP in the Contract Documents.

6.8.2 Lightweight (EPS) Fill

To offset the loading from the salt pile and reduce the stress change in the foundation soils along with the associated settlements, the installation of Expanded Polystyrene (EPS) lightweight fill could be considered. The lightweight fill would need to be placed below the slab, in conjunction with foundation drainage, and it would still be necessary to use piles to support the perimeter/push wall foundations.

The following design issues must be considered when evaluating the feasibility of using lightweight EPS fill at this site:

- Preliminary analysis indicates that the removal of up to about 4 m of soil below the existing ground surface / new concrete slab and replacement with EPS fill would be required to effectively offset the stress change due to the salt pile and reduce the settlements to tolerable levels;
- Given the high groundwater table at the site (i.e., located about 1 m below the existing ground surface), the buoyancy of the EPS fills in the long-term after the groundwater table comes to equilibrium following the excavation for construction, will be an issue. Preliminary calculations indicate a $FoS < 1.0$ against uplift of the concrete slab unless a minimum 2 m thick layer of salt was maintained on the slab at all times; and,
- A deep/pile foundation would still be required for the perimeter wall footings.

Based on the measured groundwater levels at the site, and the results of the buoyancy (uplift) analysis carried out as part of this assessment, the use of EPS lightweight fill to offset the loading from the salt pile and minimize the long-term settlements is not a practical solution. An anchoring system could be designed to resist the uplift forces from the groundwater table; however, the cost of such a system, in conjunction with the high cost of the EPS fill will make this option impractical.

6.8.3 Ground Improvement

Intrusive ground improvement method(s) could be considered to strengthen/stiffen the clayey foundation soils at the site to reduce settlements to tolerable levels. Typically, intrusive ground improvement methods consist of physically altering the properties of the compressible soils to enhance stability (if required) and reduce post-construction settlement.

The main advantage of using an intrusive ground improvement method to strengthen/stiffen the cohesive deposit is that it can be used as a stand-alone option for reducing the post-construction settlement and that typically, once completed, construction can proceed without a need for any preloading time. The main disadvantage of using a ground improvement technique is the high cost of the proprietary design and installation. Further, a significant part of the ground improvement cost is typically associated with the mobilization of equipment/plant to carry out the construction. As such, the relative cost of treating the relatively small footprints of the buildings proposed for the Arnprior MPY will be much higher than for a project where a large area of ground improvement is required.

Considering the nature of the primarily cohesive soils at the MPY site (including the water content and the liquid limit of the clayey soils), the most feasible alternatives for ground improvement considered are:

- Rigid or semi-rigid inclusions (RIs); and,
- Aggregate Piers.

Although ground improvement methods could be considered at this site, given the relatively small footprint of the MSB and VMB, the costs are anticipated to be significantly higher than the preload/surcharge option for reducing the post-construction settlements. Further, a disadvantage of these methods at this site is the requirement to penetrate through the very stiff, over-consolidated crust to improve the less-stiff underlying clay/silty clay stratum; given the thickness and stiffness of the crust, the installation of RIs or aggregate piers may be difficult at this site.

6.8.3.1 Rigid or Semi-Rigid Inclusions (RIs)

Rigid or semi-rigid inclusions are typically used at sites with higher loads over weaker soils. The inclusions are individual elements that are constructed to penetrate the weak/compressible soils and the spacing of the elements is determined by considering the magnitudes of the loads requiring support as well as the design of the load transfer platform (LTP) required to transfer the loads to the top of the elements. The LTP could be a geogrid reinforced, granular layer (i.e., incorporated into the site grade raise) or could be a reinforced concrete slab that could be designed in conjunction with the building(s) floor slab.

Semi-rigid “soilcrete” columns or inclusions could be constructed by dry soil mixing (DSM) a binder agent (typically cement) with the clayey foundation soils and may not produce any significant cuttings/spoils. Rigid cementitious elements could be constructed by pumping concrete or grout through a mandrel pushed into the foundation soils and onto a stronger bearing stratum at depth. The choice of the type of ground improvement element and the overall design of the system, including the LTP, would be the responsibility of the specialty ground improvement contractor taking into consideration cost, soil conditions, and the design performance requirement (i.e., maximum tolerable post-construction settlement in this case).

6.8.3.2 Aggregate Piers

This method of ground improvement relies on the drilling of large (approx. 0.8 m) diameter auger holes and/or insertion of a hollow steel mandrel (i.e., the displacement method) into the compressible soil strata followed by ramming lifts of granular material (such as crushed concrete or crushed stone) progressively up the hole to create a pattern of buried “aggregate piers which enhance the strength and stiffness of the treated soil mass.

We understand that the equipment readily available to perform this type of work is limited to creating pier depths of up to about 7 m to 12 m and the type of method (i.e., open auger hole vs. hollow steel mandrel) and practicality of its use will be a function of the strength/stiffness of the soil requiring improvement. Given that the clay stratum is up to about 15 m below the existing ground surface at this site, it may be necessary to consider a design that does not improve the full depth of the compressible stratum.

6.8.4 Preferred Settlement Mitigation Alternatives

From a foundation’s perspective, and following discussions with Dillon and the MTO, the preferred settlement mitigation alternatives for this site consist of the following:

- Construct the proposed grade raise across the whole site using compacted granular fill.

At the Vehicle Maintenance Building:

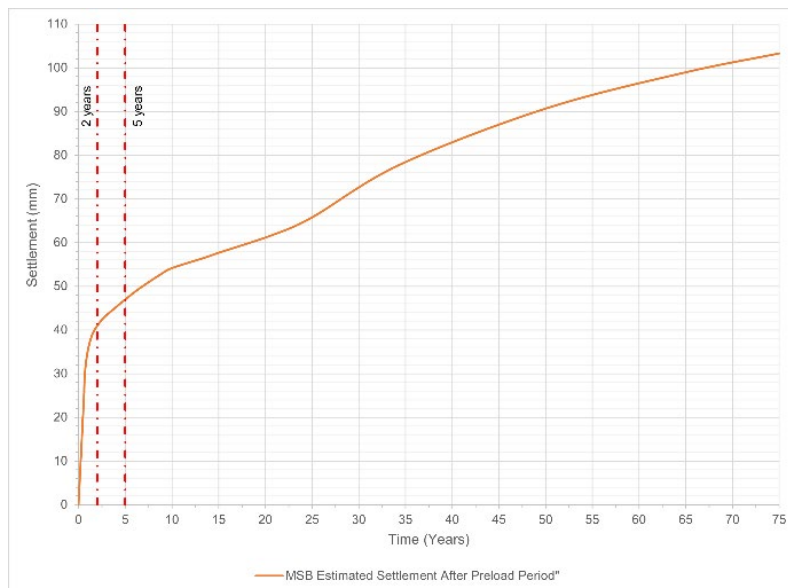
- Construct a 1 m thick surcharge fill on top of the grade raise over the footprint of the VMB as described in Section 6.8.1, extending 3 m beyond the perimeter of the building in all directions and then tapering down with 1H:1V side slopes.
- Monitor preload/surcharge settlement(s) due to the grade raise and surcharge loadings within the footprint for a seven-month preload time. See Section 6.9 for the details of the recommended instrumentation and monitoring plan.
- Remove the surcharge and re-use the granular fill in other areas of the site.
- Construct the VMB limiting the contact stress on the footings to 100 kPa at SLS.

- The estimated post-construction settlements of the VMB footings (at the 50-year and 75-year design life(s)) with these mitigation measures are shown in Figures F7 and F8 provided in Appendix F.

At the Material Storage Building

- Monitor preload settlement(s) due to the grade raise loading within the MSB footprint for a seven-month preload time.
- Construct the perimeter/push wall foundations supported on steel H-piles driven to bedrock.
- Construct the MSB.
- The estimated post-construction settlements below the salt and sand stockpiles (at the 50-year and 75-year design life(s)) with these mitigation measures are shown in Figures F9 to F11 provided in Appendix F.
- Maintain as close to a full salt pile as possible within the MSB for a period of two to five years.
- Continue monitoring settlement after construction and placement of the full salt pile.
- Plan on maintenance of the pavement structure within the MSB (i.e., padding/asphalt overlay or possibly asphalt reconstruction) one or two times over the design service life of the structure.

Repair of the flexible asphalt pavement within the stockpile area(s) will need to be included in the maintenance schedule for the facility. Graph 2 below illustrates the estimated settlement versus time at the MSB over a 75-year design life following the completion of the seven-month preload period. Based on the predicted rate of settlement, it is anticipated that the schedule for the first repairs required due to settlement under the stockpile areas would be approximately 2 years to 5 years after the construction has been completed. However, this assumes that the salt stockpile has been stored at full height during this time period; the actual rate of settlement may be different depending on the operational salt stockpile height and the actual timing for the maintenance will need to be based on visual assessment of the performance of the pavement and the results of the long-term monitoring.



Graph 2: Estimated Settlement at the MSB

6.9 Settlement Instrumentation and Monitoring Plan

A Special Provision that describes the requirements for supply and installation of the geotechnical instrumentation required to verify the response of the foundation soils in the area of the proposed VMB of the new MPY, is provided in Appendix G. The Terms of Reference for the Foundation Engineering Specialist (FES) to carry-out the foundation monitoring program as part of the Contract Administration assignment are also included in Appendix G.

The Special Provision includes the following geotechnical monitoring instrumentation to evaluate the settlement and pore pressure response of the foundation soils due to the grade raise and surcharge construction:

- Temporary Survey Benchmarks (TBMs);
- Settlement Plates (SPs); and,
- Vibrating Wire Piezometers (VWPs).

Reference should be made to the Special Provision and FES Terms of Reference for full details on equipment type, installation and monitoring requirements for this project. The following sections provide a summary of the proposed settlement monitoring plan for the MPY.

6.9.1 Instrumentation Requirements

The instrument types, quantities and approximate locations are summarized in Table 17, shown in Drawings 4 and 5 in Appendix G, and are shown on the Contract Drawings. The final locations shall be “field fit” by the Contractor to take account of any utilities that may be present, construction operations, and safe access conditions. The minimum number and approximate locations of the TBM(s) are to be determined by the Contractor and the Contractor’s Engineer in conjunction with the Contract Administrator, the Foundation Engineering Specialist, and Surveyors such that direct sighting is possible from all instruments to at least one benchmark.

Table 17: Instrumentation Requirements

Location		Monitoring Type	Name	Quantities	Tip Elevation (m)
Northing (m)	Easting (m)				
5030649.994	318064.093	Settlement Plate	SP24-01	1	N/A
5030668.575	318084.844	Settlement Plate	SP24-02	1	N/A
5030653.398	318098.527	Settlement Plate	SP24-03	1	N/A
5030634.596	318077.799	Settlement Plate	SP24-04	1	N/A
5030659.253	318074.434	Vibrating Wire Piezometer	VW24-01	1	96.5
5030648.986	318083.627	Vibrating Wire Piezometer	VW24-02	1	96.0

6.9.2 Monitoring and Reporting Frequency

6.9.2.1 Settlement Plates (SPs)

The surveyor retained by the Contractor Administrator (CA) shall survey and obtain elevations of all settlement monitoring points (SPs) at the following time intervals:

- Three consecutive readings at least one week prior to commencement of the work (Baseline Reading).
- Once immediately prior to the start of fill placement.
- Daily during fill placement for grade raise and surcharge construction.
- At the following frequency during the surcharge period (i.e., after completion of the grade raise and surcharge construction):
 - Once per week for the first 2 months;
 - Once every 2 weeks from 2 months to 4 months;
 - Once every month from 4 months to 7 months.

The FES shall submit a monitoring progress report containing the results from the SP survey to the CA weekly during the grade raise and surcharge construction and monthly until surcharge removal. Each report shall include plots of settlement versus time (in linear time and in log time) at each SP; plots of fill height versus time at each SP; and plan view, section and profile sketches of approximate fill extents at time of SP survey.

6.9.2.2 Vibrating Wire Piezometers

The FES shall collect the data recorded by the VWP dataloggers at the same frequency indicated above for the SPs. The VWP dataloggers will be programed to record the pore pressures at the VWPs once every 1 hour (24 times per day).

The FES shall submit a monitoring progress report containing the results from the VWPs to the CA weekly during the grade raise and surcharge construction and monthly until surcharge removal. Each report shall include plots of piezometric head (elevation), background piezometric head (elevation), and fill elevation versus time at each VWP; plots of excess pore pressure (EPP) and embankment vertical effective stress versus time at each VWP; and plan view, section and profile sketches of approximate fill extents at the time the VWP readings were downloaded.

6.10 Stormwater Management Pond

6.10.1 General

As discussed in Section 3.0, three boreholes were advanced within the footprint of the proposed Stormwater Management Pond (SWMP) that will be constructed as part of the Arnprior MPY development. It is understood that the SWMP is being designed as a dry pond. The existing ground surface at the SWMP boreholes ranges in elevation from 104.4 m to 103.8 m. As outlined in Section 4.0 and shown in Drawing 2, the subsurface conditions encountered in the boreholes advanced for the SWMP generally consist of a 4.3 m thick, stiff to very stiff weathered, clay crust overlying a 5.0 m thick firm to stiff clay/silty clay, over a thin layer of till, underlain by marble bedrock. The depth to bedrock at the location of the SWMP was confirmed by coring at Borehole 23-03 at a depth of 9.4 m below existing grade (Elevation 94.6 m).

The level of the groundwater was measured in the monitoring well at Borehole 23-03 at Elevations 103.2 m and 98.8 m on October 19 and December 12, 2023, respectively. Table 5 summarizes the groundwater levels measured at the site.

Based on the drawings provided by Dillon, only a nominal grade raise is required across the footprint of the SWMP (i.e., about 0.1 m); however, larger grade raises are required to construct the perimeter berms around the dry pond. It is our understanding that, in general, no significant cuts below existing ground surface are required to achieve the design grades of the SWMP.

The design details for the proposed SMWP are provided in the following drawings from the project's 60% Drawing submission package dated November 23, 2023.

- *Sheet 27 – SWM Pond Key Plan Arnprior MPY*
- *Sheet 28 – SWM Pond Sections Arnprior MPY*

Based on the information in these drawings, the proposed pond is to be located approximately 80 m southwest of the VMB and is approximately 95 m wide and 200 m long. The designed 100-year water level is set at an Elevation of 104.35 m. The side berms are to be constructed with 4H:1V side slopes with an approximate height of about 1.5 m above the existing grade. Construction drawings for the SWMP indicate that the base of the pond is to be constructed essentially at existing grade with only localized excavation required for the construction of the *Low Flow Channel* within the SWMP set at Elevation 103.5 m at the lowest point near the outlet of the structure or approximately 0.5 m below existing grade.

The proposed SWMP footprint and profile details are shown in Drawing 2

6.10.2 Discussion and Engineering Recommendations

6.10.2.1 Base Stability and Uplift Pressure

Basal heave due to hydrostatic uplift pressures can result when only a limited thickness of low-permeability soil (e.g., clay) beneath the base of an excavation is underlain by higher permeability soil or fractured bedrock which can result in a disturbed/destabilized subgrade and slope failures, during excavation activities. Basal heave can also cause construction issues (disturbed subgrades, poor trafficability, etc.).

Given that only a minor, localized excavation (less than 1.0 m deep below the existing ground surface) will be required to construct the low-flow channel through the pond at this site with a base of cut elevation (at Elevation 103.5 m) above the groundwater level as measured in Borehole 23-03 (i.e., at Elevation 103.2 m on October 19, 2023), there would be no risk of base heave occurring. An additional check-analysis was carried out considering the effect of the groundwater being at the ground surface at the time of construction and for this condition, the results of the basal heave analysis indicate that, within the pond footprint, there would be a factor of safety (FoS) against basal heave of 1.9 for excavations that extend to a maximum grade of approximately Elevation 103.5 m.

6.10.2.2 Assessment of Global Stability

The global stability for the pond side slopes was evaluated using limit equilibrium analysis and the GeoStudio 2023.1.0 Slope/W software. The geometry used in the stability analysis was based on the topographic survey for the site as well as the available information for the proposed pond.

As noted in Section 6.6.1 for the stability analyses, target minimum FoS of 1.33 and 1.54 has been used for the design of the berm slopes, considering global stability for temporary (short-term) and permanent (long-term) conditions, respectively, as per Table 6.2 of CHBDC (2019).

The following geometry/parameters were used in the analysis:

- Pond side slopes at 4 Horizontal to 1 vertical;
- Material properties as summarized in Table 14;
- Maximum berm height of up to about 1.5 m above existing grade;
- Soil stratigraphy based on Section C-C shown in Drawing 2; and,
- Groundwater level at Elevation 103.2 m.

The following three scenarios/conditions were considered in the stability analyses of the pond excavation side slope:

- End of Construction (short-term conditions, critical/deepest stage of excavation) assuming undrained shear strengths and total stress conditions; and,
- Steady-state long-term drained shear strengths and effective stress seepage conditions under the following groundwater conditions:
 - pond water at 1:100-year water Elevation 104.35 m (60% Drawing submission package Sheet 28); and,
 - pond empty.

The results of the short-term/total stress stability analysis indicate that the FoS is greater than 1.3 for a deep-seated slip surface that could affect the stability of pond side slopes. For the long-term/effective stress stability analysis for both an empty and full pond, the FoS is greater than 1.5 for a similar deep-seated slip surface.

6.10.2.3 Site Preparation and Excavation for SWMP

The site preparation and excavation recommendations are outlined below in Section 6.11 will apply for the construction of the SWMP.

6.10.2.4 Dewatering

It is our understanding that Dillon is responsible for evaluating the hydrogeological aspects of the SWMP pond design. Since the base elevation of the bottom of the pond is near the existing ground surface and only localized excavation up to less than 1.0 m below the existing ground surface is required to construct the low-flow channel (which is above the groundwater level measured on October 19, 2023), dewatering is not anticipated to be required prior to and during the SWMP pond construction. If the groundwater table is higher at the time of construction and if dewatering is required, it is recommended that an MECP-licensed, specialist dewatering subcontractor supervise the installation, operation, and decommissioning of all dewatering systems for this project, in accordance with applicable legislation.

The method of construction dewatering is to be solely determined by the Contractor based on their own assessment of the site-specific conditions, and likely by their specialist dewatering contractor.

We note that according to O.Reg. 63/16 and O.Reg. 387/04 if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 L/day and less than 400,000 L/day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a "Water Taking Plan". Alternatively, a Permit to Take Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 L/day is to be pumped from an excavation.

6.10.2.5 Berm Materials

The pond berms may be constructed using engineered fill consisting of suitable earth borrow (free of organics/deleterious material) meeting the requirements of OPSS.PROV 212 (Earth Borrow) or OPSS.PROV 1010 Select Subgrade Material (SMM), Granular 'A', or Granular 'B' (Type I or II). All fill materials should be placed in a maximum of 300 mm thick loose lifts in accordance with OPSS.PROV 206 (Grading) and be compacted in accordance with OPSS.PROV 501. Earth fill shall be compact to at least 95% of the material's standard Proctor maximum dry density (SPMDD) and granular fill materials shall be compacted to at least 98% of the material's SPMDD.

It is noted that the low permeability native clay deposit at the base of the pond could promote preferential drainage through the soils comprising the berms around the pond if the berm(s) are constructed of coarse-grained soils (i.e. granular engineered fill as discussed above). Mitigation for this condition could consist of construction of a lower-permeability liner (i.e., compacted clay liner or Geosynthetic Clay Liner (GCL)) on the interior berm slopes keying into the underlying clay deposit to mitigate a preferential drainage path under the berms. Alternatively, the berm(s) could be constructed with a natural fine-grained, low permeability material which wouldn't necessitate the use of a liner, so long as the berm material is keyed into the native clay subgrade.

If a GCL is used to prevent preferential drainage through the berm material it should be anchored/keyed in behind the crest of the berm(s) and into the native clay at the toe(s) and covered by a minimum 300 mm thickness of fill to protect it from damage from construction equipment, or degradation from exposure to sunlight during its operational life. Below the high-water level, this protection fill should consist of OPSS.PROV 1010 Granular 'A' or Granular 'B' (Type I or II) material. Above the high-water level, the protection fill may consist of earth fill and topsoil to promote vegetation growth; however, at this site it is acknowledged that the high-water level is near the top of the berm(s) and hence, it is likely most practicable to place granular fill materials over the liner along the full interior slope of the berm. The protective fill should be placed in a maximum 300 mm thick layer and nominally compacted by the construction equipment.

If a compacted clay liner is adopted, the natural clay soil material should meet the requirements of OPSS.PROV 1205. The clay liner should be keyed-in to the native clay at the toe of the berm(s) and placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the material's SPMDD as per OPSS.PROV 501. The natural clay liner should be covered with at least 300 mm of fill consisting of OPSS.PROV 1010 Granular 'A' or Granular 'B' (Type I or II) material to provide protection from sunlight, desiccation, and possible damage during periodic pond clean-out operations.

6.11 Construction Considerations

6.11.1 Open Cut Excavations

All excavations at the site should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Excavations to depths of up to about 2 m through the grade raise fills and possibly into the top of the native clay crust are anticipated to be required at some locations to construct footings and pile caps with appropriate frost cover. The groundwater level at the site has been measured as high as Elevation at 104.0 m (i.e., within the weathered clay crust), in some case above the anticipated founding level.

The soils at this site would be generally classified as Type 3 soils (compact to loose fill material and stiff native clay) in accordance with the OHSA. Accordingly, temporary excavations should be made with side slopes no steeper than 1H:1V.

6.11.2 Groundwater and Surface Water Control

Based on excavating to approximately 2.0 m below top of grade raise fills for construction for footings and piles caps at the VMB and MSB, and up to approximately 1.0 m below the existing ground surface at the SWMP, dewatering is not anticipated to be required at the site for construction.

Any groundwater inflows should be limited and likely can be handled by pumping from sumps within the excavations based on the soil and groundwater conditions at the site. However, the selection and design of temporary unwatering/dewatering system is the responsibility of the Contractor.

6.11.3 Subgrade Preparation and Backfilling

Prior to constructing the grade raise on the site, all existing fills and topsoil/organic soils should be stripped and removed from the site and the subgrade proof-rolled and inspected by a geotechnical engineer. Any soft areas should be sub-excavated and removed and replaced with compacted granular fill as part of the grade raise construction. The grade raise should be constructed with OPSS.PROV 1010 Granular A or Granular B Type II fill material placed and compacted as an engineered fill.

At the completion of the preload period, excavation to the bottom of footing/pile cap founding elevation should be carried out and the subgrade inspected prior to footing construction. As outlined in Section 6.4, the native subgrade for the footings should be either compacted Granular A or Granular B Type II grade raise fill or native very stiff weathered clay crust (at elevations not lower than approximately Elevation 103 m).

The native clay subgrade will be easily disturbed during wet weather and under winter conditions and should be protected with a concrete working slab promptly after excavation and inspection. The exposed subgrade could be covered with a 100 mm thick concrete working slab. After the concrete for the working slab has set, the footings could then be constructed directly on the working slab without the need for a granular pad or bedding material. Suggested wording for an NSSP to alert the Contractor to the requirement for a working slab has been provided in Appendix G.

The existing fill materials and natural soils at this site are considered frost susceptible and should not be used as backfill against foundation walls. To avoid problems with frost adhesion and heaving, the foundation walls should be backfilled with non-frost susceptible sand or sand and gravel conforming to the requirements for OPSS.PROV 1010 Granular B Type I or II.

To avoid ground settlements around the foundations, which could affect site grading and drainage, all the backfill materials should be placed in maximum 300 mm thick loose lifts and be compacted to at least 95% of the materials Standard Proctor Maximum Dry Density (SPMDD) in accordance with OPSS.PROV 501.

The foundation wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump pit from which the water is pumped.

6.12 Corrosion and Cement Type

One groundwater sample and two soil samples were submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The test results are provided in Appendix C.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results in Table 6 and Table 7 provided in Section 4.5 of this report, were compared with Table 3 of Canadian Standards Association Standards A23.1-19 (CSA A23.1) and generally indicate a low degree of sulphate attack potential on concrete structures at this site. Accordingly, GU cement could be specified for concrete in below grade applications.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Generally, the test results provided in Table 6 and Table 7 indicate a moderate to high potential for corrosion of exposed ferrous metal at the site which should be considered in the design.


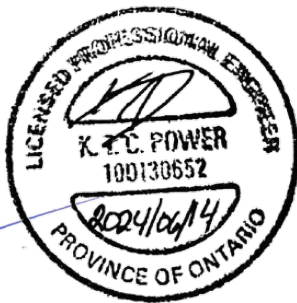
7.0 CLOSURE

This report was prepared by Kinjal Gajjar, Geotechnical Consultant, and Kenton Power, P.Eng. The report was reviewed by Paul Dittrich, P.Eng. a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP conducted an independent technical and quality review of this report.


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KG/KCP/JPD/yj

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ASTM International

- ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
- ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil

Ontario Provisional Standard Drawing:

OPSD 3090.101 Foundation Frost Penetration Depths for Southern Ontario

Ontario Provincial Standard Specification:

OPSS PROV 206 Construction Specification for Grading

OPSS.PROV 212 Construction Specification for Earth Borrow

OPSS.PROV 501 Construction Specification for Compacting.

OPSS.PROV 902 Construction Specification for Excavating and Backfilling - Structures

OPSS.PROV 903 Construction Specification for Deep Foundations

OPSS.PROV 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill
Material

OPSS.PROV 1205 Material Specification for Clay Seal

Table 18: Comparison of Foundation Alternatives

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Steel H-piles driven to bedrock	<ul style="list-style-type: none">Feasible and preferred for support of Material Storage Building (MSB) perimeter/push walls.Feasible but not recommended for the Vehicle Maintenance Building (VMB) foundations based on the relatively low design loads	<ul style="list-style-type: none">Conventional construction methods.Higher geotechnical axial resistance, compared to spread footings.Increased lateral resistance at perimeter/push walls.Negligible postconstruction settlement.	<ul style="list-style-type: none">More expensive than shallow foundation option, but a deep foundation solution may be the only option if settlements can't be tolerated in the design.	<ul style="list-style-type: none">Negligible risk of postconstruction settlement of the MSB foundations.Low risk of not sufficiently penetrating through the till and onto the bedrock surface.	<ul style="list-style-type: none">Lower cost than drilled shafts (large diameter caissons).Much higher cost than shallow foundations.
Drilled Shafts (Caissons)	<ul style="list-style-type: none">Feasible but not recommended for support of VMB or MSB foundations.	<ul style="list-style-type: none">Higher geotechnical axial resistance compared to spread footings and driven piles.Higher lateral resistance than driven piles, if required for perimeter/push walls.Reduced number of deep foundation elements compared to steel H piles, although thicker/stronger pile caps may be required between pile elements.Negligible post-construction settlement.	<ul style="list-style-type: none">Potential for squeeze or base heave as the caisson is advanced through the clayey strata given the high-water table and granular (till) materials above the bedrock.Permanent liners may be required for construction to mitigate against soil squeeze and base heave.Concrete would have to be placed by tremie methods below the water level.Cleaning and inspection of the base below the water table could be difficult.Greater risk of encountering obstructions due to larger size of drill hole required.	<ul style="list-style-type: none">Negligible risk of post-construction settlement of the MSB foundations as long as the base of caissons is properly cleaned and inspected.	<ul style="list-style-type: none">Much higher cost than shallow foundations.Higher cost than driven steel H-piles.Installation cost could be impacted by the need for a liner to minimize disturbance and loss of ground and for tremie concrete placement.
Material Storage Building (MSB) Cast-in-place or precast spread footings supported on compacted granular grade raise fill over native very stiff weathered clay crust.	<ul style="list-style-type: none">Likely not feasible due to thick, cohesive deposits resulting in low geotechnical resistance(s) and large post-construction settlements caused by loading from the adjacent interior salt stockpile.	-	-	-	-
Vehicle Maintenance Building (VMB) Cast-in-place or precast spread footings supported on compacted granular grade raise fill over native very stiff weathered clay crust.	<ul style="list-style-type: none">Feasible above approximate Elevation 103 m based on a design footing load (contact stress) of 100 kPa at SLS for 25 mm; no large stockpiles/heavy loads causing excessive settlement.	<ul style="list-style-type: none">Conventional construction methods.Anticipated loading is such that ground improvement is not required.	<ul style="list-style-type: none">Concrete working slab is required due to the potential for the native clay subgrade to be disturbed during construction.	<ul style="list-style-type: none">Over excavation of the footing trench causing insufficient clay crust below the footing to provide adequate bearing resistance.	<ul style="list-style-type: none">Lowest cost option compared to deep foundation options.

Table 19: Comparison of Settlement Mitigation Alternatives for Material Storage Building (MSB)

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Preloading (7 months) using the site grade raise fill for preload.	<ul style="list-style-type: none">Feasible and recommended for reduction of post-construction settlement below the salt pile floor slab.	<ul style="list-style-type: none">Reduces post-construction settlement and therefore reduces amount of post-construction maintenance to pavement structure below salt stockpile.Since grade raise material is being placed as part of site grading, no additional material would be required to construct the preload.No additional costs required for “double handling” of the preload fill material since preload fill material would not need to be removed from footprint prior to commencing construction of MSB foundations.	<ul style="list-style-type: none">Preloading to reduce post-construction settlements to tolerable amounts requires at least 3 years; not as fast as surcharge or wick drain options and is likely impractical for typical construction schedules.Preload duration of 7 months (that can be accommodated in the construction schedule) will result in post-construction settlements that require future maintenance of asphalt pavement in the salt stockpile area.Would require settlement and pore pressure monitoring program during fill placement and during preload period.Construction schedule delayed while the preloading period is carried out.	<ul style="list-style-type: none">Results of the monitoring program may indicate that the rate of consolidation settlement is slower than predicted; in this case, additional post-construction maintenance (i.e., padding, or pavement reconstruction) could be required.	<ul style="list-style-type: none">Additional costs required for instrumentation and monitoring.Additional costs for post-construction maintenance to repair asphalt pavement in the salt stockpile area.
Pile Support of Perimeter/Push Walls and Use Salt Stockpile as Preload (Stockpile area to incorporate flexible pavement in design plus planned future repair/reconstruction in maintenance schedule)	<ul style="list-style-type: none">Feasible – best option if the construction schedule cannot accommodate the required duration of the preload/surcharge option (see below).	<ul style="list-style-type: none">No construction schedule delay as a preloading/surcharge period is not required.Reduced material handling costs.Settlement monitoring program not necessarily required.	<ul style="list-style-type: none">Increased cost for planned repair/reconstruction of flexible pavement in salt stockpile area as part of maintenance schedule at least once within the first 2 to 5 years of operation.Operational Constraint or NSSP required to direct AMC to maintain salt stockpile height at or as close to full height as possible for the first few years of operation (in particular during non-winter months).	<ul style="list-style-type: none">Negligible risk of postconstruction settlement of the material storage building foundation(s) since supported on piles.Potential operational issues associated with settlements in the salt stockpile area until repair/reconstruction of flexible pavement is carried out.	<ul style="list-style-type: none">Less expensive than some mitigation measures such as EPS lightweight fill, and ground improvement techniques such as aggregate piers or rigid inclusions.

Comparison of Settlement Mitigation Alternatives for Material Storage Building (MSB) cont'd.

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Preloading + Surcharging with Wick Drains	<ul style="list-style-type: none">Not recommended.	<ul style="list-style-type: none">Accelerates settlements during preload/surcharge period.Reduces time period for preloading/surcharging to 4 to 8 months.	<ul style="list-style-type: none">Accelerates swelling/rebound during removal of the surcharge as required to start facility construction (due to the over-consolidated nature of clay stratum). Depending on the pace of construction, settlements that occurred during the preload/surcharge period may be offset by subsequent swelling/rebound, and then post-construction settlements will still occur.Very stiff weathered crust may require pre-drilling at some locations to facilitate penetration of the wick drains.Would require settlement and pore pressure monitoring program during fill placement and during preload/surcharge period.Additional time required to install wick drains.Surcharge material will need to be removed from footprint prior to commencing construction.	<ul style="list-style-type: none">Reduced effect of preload/surcharge due to the over-consolidated nature of clay stratum results in some amount of post-construction settlement still occurring.Results of the monitoring program may indicate that the end of primary consolidation is not reached at the end of the estimated preload/surcharge period thereby further increasing delay in construction; however, the risk is less than for preload/surcharge options without wick drains.	<ul style="list-style-type: none">Additional costs required for instrumentation and monitoring.Increased cost of additional surcharge material.Additional costs required for “double handling” of the preload/surcharge fill material.Additional costs for wick drain installation, and possibly additional costs for pre-drilling for wick drains to penetrate thick, very stiff weathered crust.
Lightweight (EPS) fill under the Salt Stockpile	<ul style="list-style-type: none">Not feasible as expanded polystyrene (EPS) fill will eventually become buoyant when the full hydrostatic groundwater pressures come to equilibrium post-construction.High risk that the uplift forces will be sufficient to lift the floor slab unless a minimum 2 m thick layer of salt is maintained on the floor slab at all times; or alternatively, an anchoring system is provided to resist the uplift forces on the EPS.				
Ground Improvement (Rigid or semi-Rigid Inclusions or Aggregate Piers)	<ul style="list-style-type: none">Feasible for reduction of post-construction settlement.Not considered to be practical due to high costs of ground improvement for the relatively small footprint at this site as compared to other alternatives.	<ul style="list-style-type: none">Reduces post-construction settlement to tolerable levels.Minimal waiting period required after completion of ground improvement prior to start of construction.No additional fill required as in the case of preload/surcharging.Could eliminate the need to support the perimeter/push walls on pile foundations.	<ul style="list-style-type: none">Proprietary design and installation by specialty ground improvement contractor.The native clay material at the site and the settlement tolerance may require use of cementitious (rigid or semi-rigid) elements resulting in higher costs.Thickness of the clay may mean that aggregate piers may not be feasible at this site. Further consultation with the speciality contract would be required.	<ul style="list-style-type: none">Additional investigation and testing (potentially including bench-scale testing) may be required for detail design.Delay in schedule to design and construct ground improvement.	<ul style="list-style-type: none">Much higher cost than other mitigation options due to additional costs for construction using specialised equipment.Additional cost for investigation and testing may be required for detail design of the ground improvement program.

Table 20: Comparison of Settlement Mitigation Alternatives for Vehicle Maintenance Building (VMB)

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Preloading (7 months) using the site grade raise for preload + Surcharging (approximately 1 m high extending to 5 m beyond the perimeter of the footprint of VMB)	<ul style="list-style-type: none">Feasible – best option if construction schedule can accommodate the recommended 7-month preload/surcharge period and assuming a maximum footing load (contact stress) of 100 kPa at SLS.	<ul style="list-style-type: none">Reduces post-construction settlement of the footings to about 25 mm or less.Since grade raise material is being placed as part of the construction grading plan, only surcharge material would be additional material required.Granular surcharge fill could be re-used for construction at other areas of the site.	<p>Additional costs are required for “double handling” of the surcharge fill material.</p>	<ul style="list-style-type: none">Results of the monitoring program may indicate that sufficient consolidation does not occur within the estimated preload period thereby further increasing delay in construction.	<ul style="list-style-type: none">Additional costs required for instrumentation and monitoring.Additional costs surcharge material.
Preloading using the site grade raise fill for preload.	<ul style="list-style-type: none">Feasible for reducing post-construction settlement of the shallow foundations of the VMB to tolerable levels.	<ul style="list-style-type: none">Reduces post-construction settlement to tolerable levels.Since grade raise material is being place as part of the construction grading plan, no additional material would be required to construct the preload.No additional costs required for “double handling” of the preload fill material preload fill material would not need to be removed from footprint prior to commencing construction.	<ul style="list-style-type: none">Preload duration to reduce post-construction settlements to less than 25 mm (assuming a footing load/contact stress of 150 kPa) estimated to be 2 to 3 years; likely impractical for typical constructions schedule.Would require settlement and pore pressure monitoring program during fill placement and during preload period.Construction schedule delayed while settlement occurs.	<ul style="list-style-type: none">Results of the monitoring program may indicate that the consolidation is not reached at the end of the estimated preload period thereby further increasing the delay in construction.	<ul style="list-style-type: none">Costs for delay in the construction schedule.Additional costs required for instrumentation and monitoring.

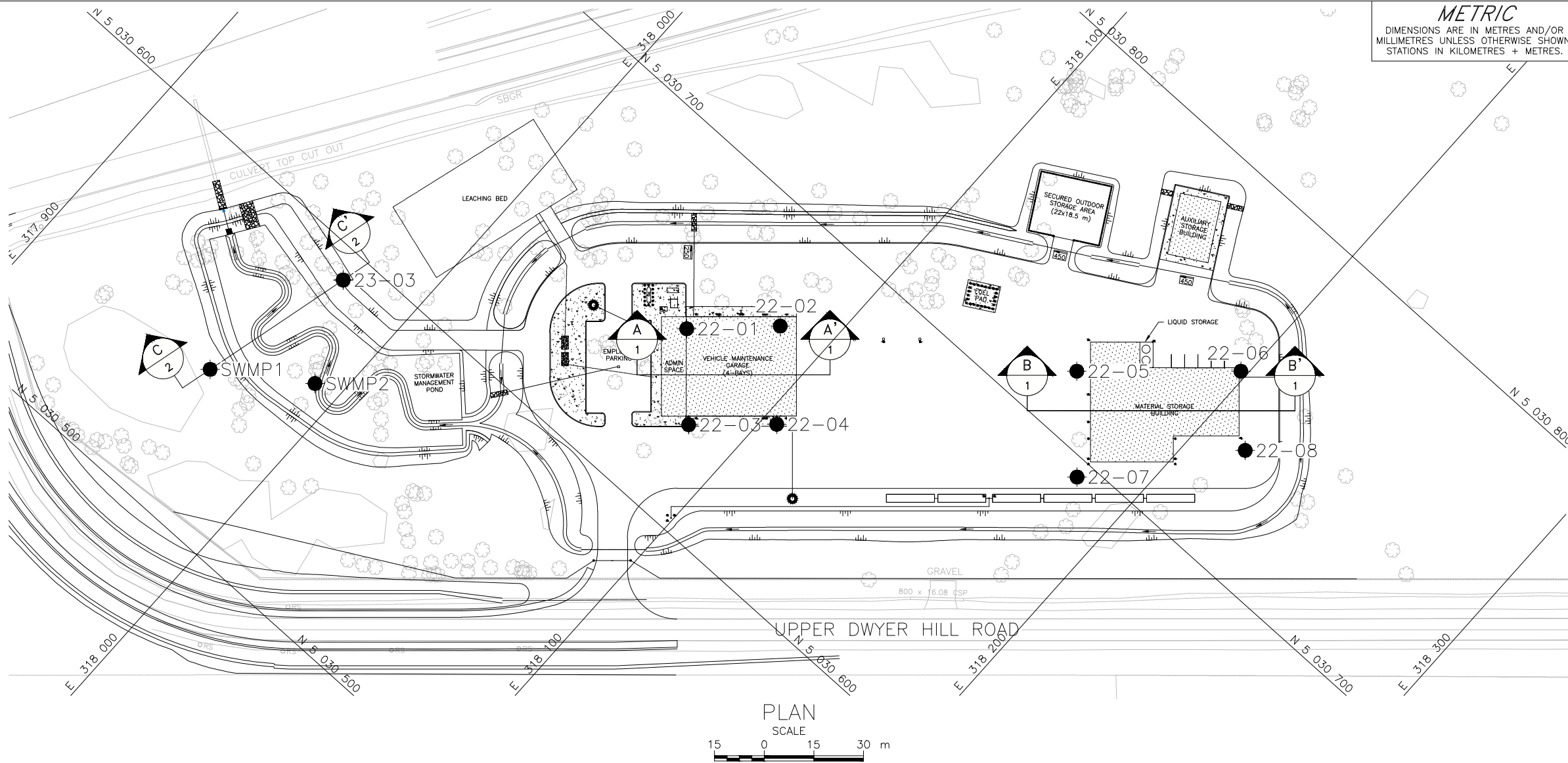
Comparison of Settlement Mitigation Alternatives for Vehicle Maintenance Building (VMB) cont'd.

Option	Feasibility	Advantages	Disadvantages	Risks/Consequences	Relative Costs
Preloading + Surcharging with Wick Drains	<ul style="list-style-type: none">■ Not recommended.	<ul style="list-style-type: none">■ Accelerates settlements during preload/surcharge period.■ Reduces time period for preloading/surcharging to 4 to 8 months.	<ul style="list-style-type: none">■ Accelerates swelling/rebound during removal of surcharge as required to start facility construction (due to over-consolidated nature of clay stratum). Depending on pace of construction, settlements that occurred during preload period may be offset by subsequent swelling/rebound and then post-construction settlements will still occur.■ Very stiff weathered crust may require pre-drilling at some locations to facilitate penetration of the wick drains.■ Would require settlement and pore pressure monitoring program during fill placement and during preload/surcharge period.■ Additional time required to install wick drains.■ Surcharge material will need to be removed from footprint prior to commencing construction.	<ul style="list-style-type: none">■ Reduced effect of preload/surcharge due to over-consolidated nature of clay stratum results in some amount of post-construction settlement still occurring.■ Results of the monitoring program may indicate that the end of primary consolidation is not reached at end of estimated preload/surcharge period thereby further increasing delay in construction; however, risk is less than for preload/surcharge options without wick drains.	<ul style="list-style-type: none">■ Additional costs required for instrumentation and monitoring.■ Increased cost of additional surcharge material.■ Additional costs required for “double handling” of the preload/surcharge fill material.■ Additional costs for wick drain installation.■ Additional costs for wick drain installation, and possibly additional costs for pre-drilling for wick drains to penetrate thick, very stiff weathered crust.

DRAWINGS

Drawings 1 and 2 – Borehole Locations and Soil Strata

Drawing 3 – Grade Raise Contour Plan



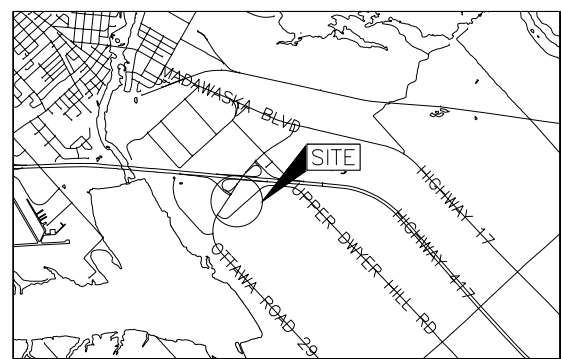
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2024-4033
GWP No. 4024-22-00

HIGHWAY 417 AT OTTAWA ROAD 29
ARNPRIOR MAINTENANCE PATROL YARD
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
134



KEY PLAN
SCALE

1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- ⊢ Seal
- ⊢ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- ⊢ WL in piezometer, measured on Dec. 12, 2023

BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 9)

No.	ELEVATION	NORTHING	EASTING
22-01	104.9	5030649.6	318064.8
22-02	104.7	5030669.0	318085.3
22-03	106.7	5030628.4	318084.5
22-04	106.1	5030646.3	318104.3
22-05	105.4	5030718.7	318161.1
22-06	105.7	5030751.7	318198.0
22-07	106.3	5030694.9	318182.5
22-08	105.8	5030734.8	318215.0
23-03	104.0	5030591.3	317977.7

Structure Location: Latitude: 45.415665 ; Longitude: -76.330521

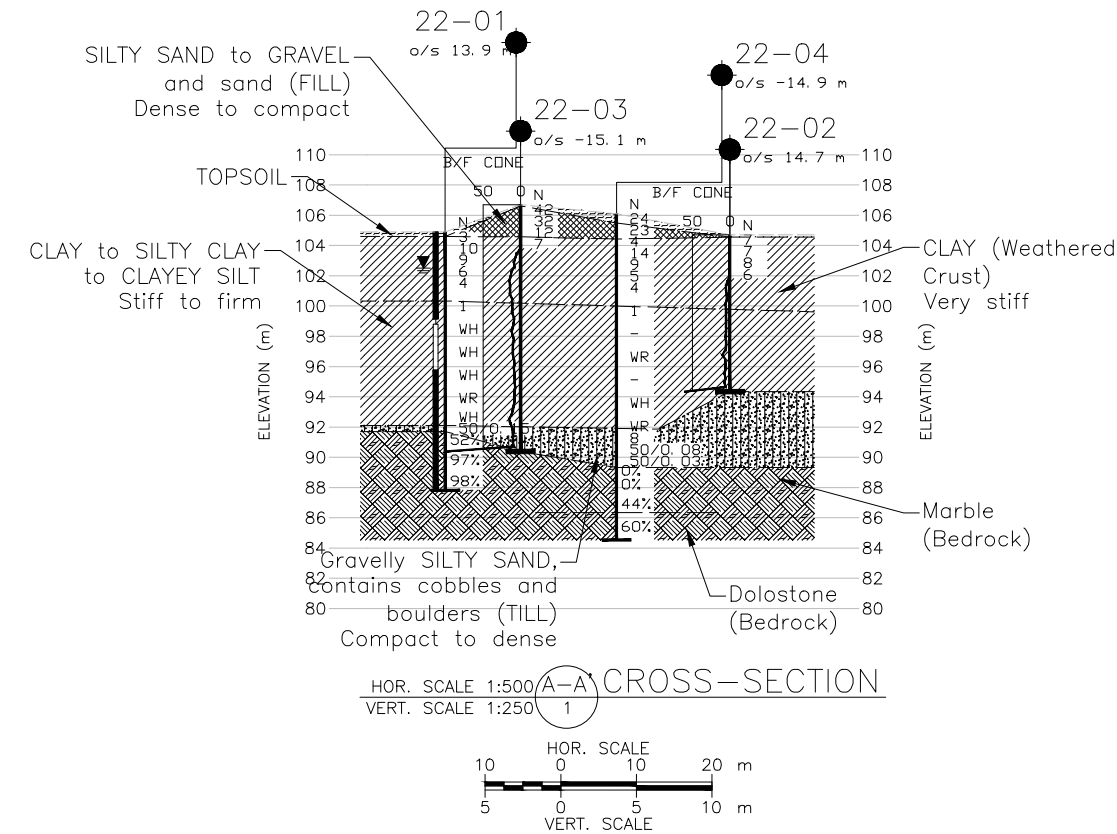
NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

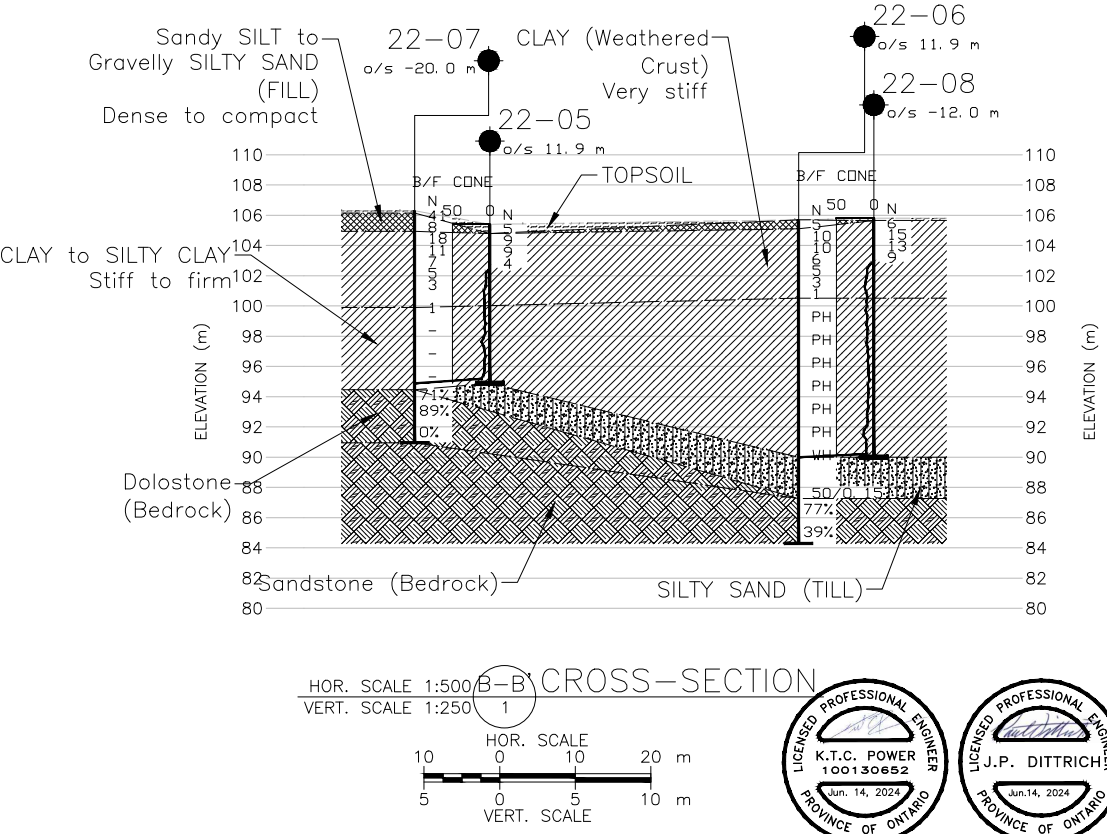
REFERENCE

1. Base plans provided in digital format by Dillon Consulting Ltd., drawing file no. 4059 - CAD for Golder.dwg, received November 29, 2023.



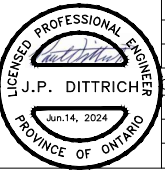
HOR. SCALE 1:500
VERT. SCALE 1:250

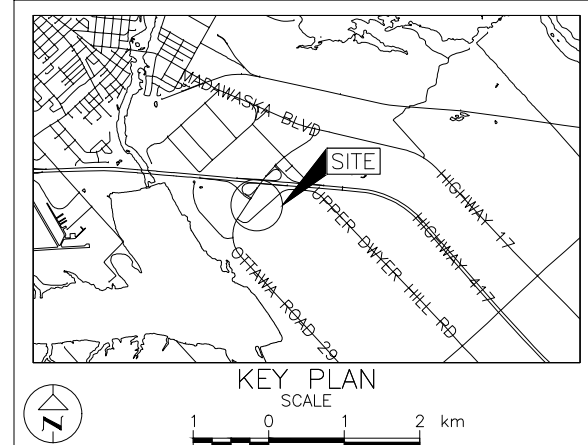
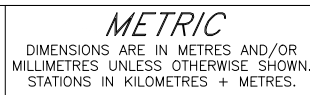
10 0 20 m
5 0 10 m
VERT. SCALE







HOR. SCALE 1:500
VERT. SCALE 1:250

10 0 20 m
5 0 10 m
VERT. SCALE





LEGEND	
	Borehole – Current Investigation
	Seal
	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
	WL in piezometer, measured on Oct. 19, 2023



BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 9)			
No.	ELEVATION	NORTHING	EASTING
23-03	104.0	5030591.3	317977.7
SWMP1	104.4	5030544.4	317965.7
SWMP2	103.8	5030562.4	317992.2

Structure Location: Latitude: 45.415665 ; Longitude: -76.330521

NOTES

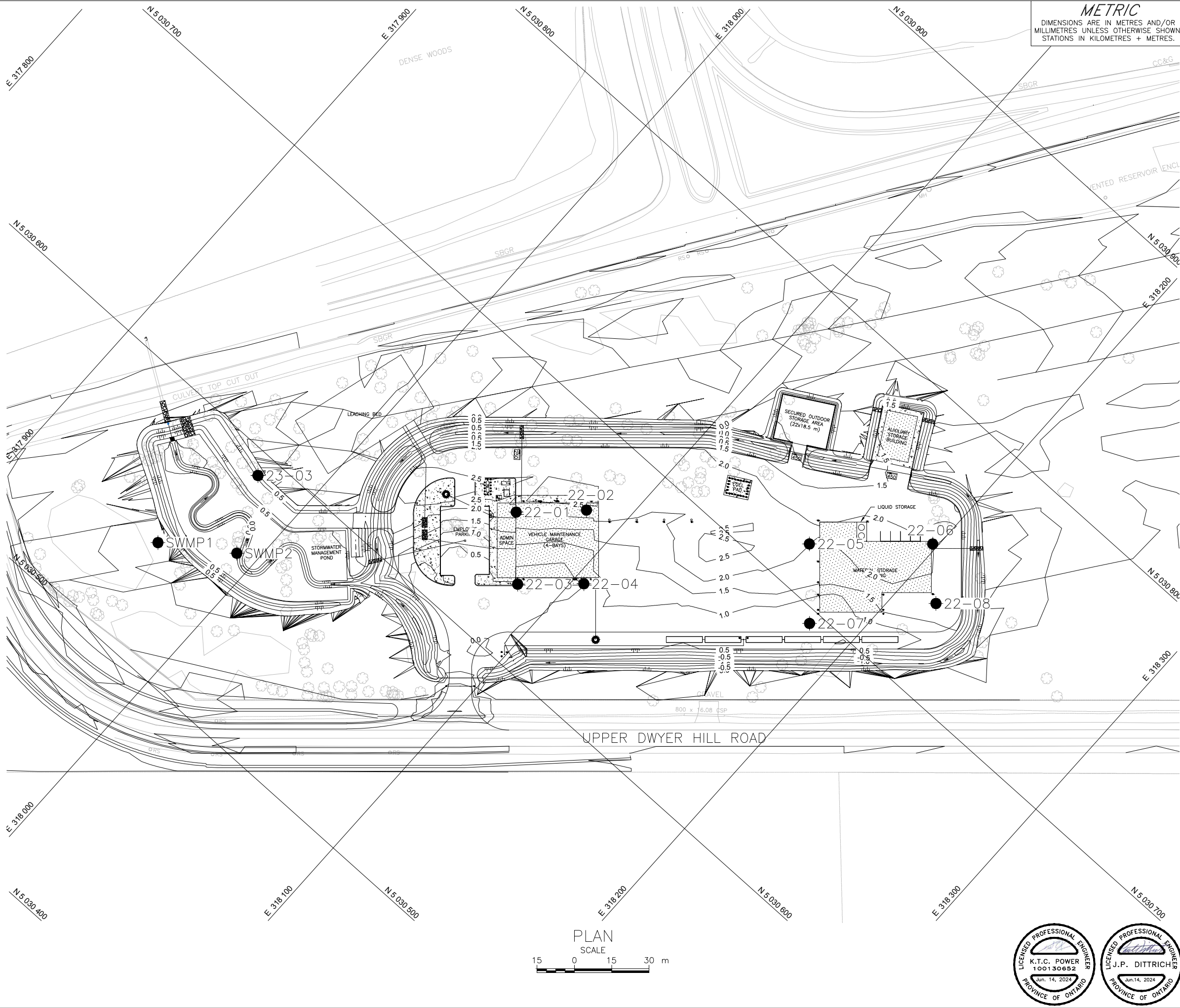
This drawing is for subsurface information only. The proposed structure details/wells are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

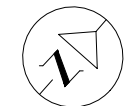
1. Base plans provided in digital format by Dillon Consulting Ltd., drawing file no. 4059 – CAD for Golder.dwg, received November 29, 2023.

NO.	DATE	BY	REVISION						
Geores No. 31F08-001									
HWY. 417			PROJECT NO. CA0012298.4565				DIST. EASTERN		
SUBM'D. KG		CHKD. KG		DATE: 6/14/2024			SITE: -		
DRAWN: JM		CHKD. KCP		APPD. JPD			DWG. 2		



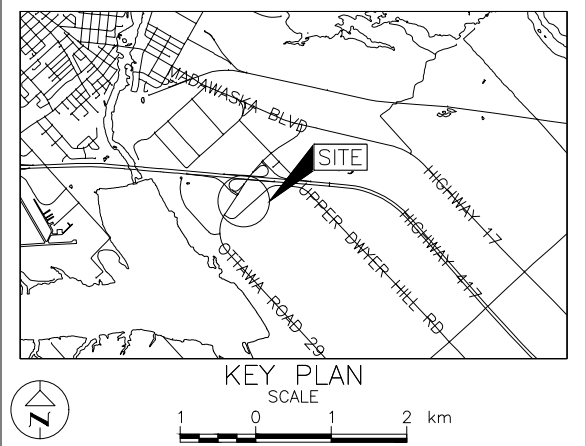
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2024-4033
GWP No. 4024-22-00



HIGHWAY 417 AT OTTAWA ROAD 29
ARNPRIOR MAINTENANCE PATROL YARD
GRADE RAISE CONTOUR PLAN

SHEET
136



BOREHOLE CO-ORDINATES (NAD 83 MTM ZONE 9)			
No.	ELEVATION	NORTHING	EASTING
22-01	104.9	5030649.6	318064.8
22-02	104.7	5030669.0	318085.3
22-03	106.7	5030628.4	318084.5
22-04	106.1	5030646.3	318104.3
22-05	105.4	5030718.7	318161.1
22-06	105.7	5030751.7	318198.0
22-07	106.3	5030694.9	318182.5
22-08	105.8	5030734.8	318215.0
23-03	104.0	5030591.3	317977.7

Structure Location: Latitude: 45.415665 ; Longitude: -76.330521

NOTES
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE
1. Base plans provided in digital format by Dillon Consulting Ltd., drawing file no. 4059 - CAD for Golder.dwg, received November 29, 2023.
2. Grade raise contour plans provided in digital format by Dillon Consulting Ltd., drawing file no. 4059-02-SRF-DES.dwg, received November 29, 2023.

PLAN
SCALE
15 0 15 30 m



Geocres No. 31F08-001			
HWY. 417		PROJECT NO. CA0012298_4565	
SUBM'D. KG	CHKD. KG	DATE: 6/14/2024	SITE: -
DRAWN: JM	CHKD. KCP	APPD. JPD	DWG. 3

APPENDIX A

Borehole Records

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes and Drill Holes 22-01 to 22-08, SWMP1, SWMP2 and 23-02

Bedrock Core Photographs, Figures A1 to A10

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (i.e., SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (i.e., some sand)
0	trace (i.e., trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \gamma_s / \gamma_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_C	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{(e)}$	secondary compression index
C	rate of secondary compression
$C_{(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint	KV Karstic Void
BD Bedding	K Slickensided
BC Broken Core	LC Lost Core
CC Continuous Core	MB Mechanical Break
CL Closed	PL Planar
CO Contact	PO Polished
CU Curved	RO Rough
CT Coated	SA Slightly Altered
FLT Fault	SH Shear
FOL Foliation	SM Smooth
FR Fracture	SR Slightly Rough
GO Gouge	SY Stylolite
IN Infilled	UN Undulating
IR Irregular	VN Vein
JN Joint	VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250



PROJECT		21480555		RECORD OF BOREHOLE		No 22-01		SHEET 1 OF 2		METRIC										
G.W.P.		4024-22-00		LOCATION		N 5030649.6; E 318064.8 MTM NAD 83 ZONE 9 (LAT. 45.415665; LONG. -76.330521)		ORIGINATED BY		KG										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring		COMPILED BY		TR										
DATUM		Geodetic		DATE		June 27, 2022		CHECKED BY		KCP										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	20	40	60	80	100	W _p	W	W _L	γ	GR	SA	SI	CL
104.9	0.0	GROUND SURFACE																		
104.6	0.3	(CL/ML) SILTY CLAY to CLAYEYS SILT (TOPSOIL) Grey-brown Moist		1	SS	3														
		(CH) CLAY (WEATHERED CRUST) Very stiff Grey-brown w>PL		2	SS	10	104													
				3	SS	9	103													
				4	SS	6	102										0	0	41	59
				5	SS	4	101													
							100													
100.3	4.6	(CI) SILTY CLAY Stiff to Firm Grey w>PL		6	SS	1	99													
				7	SS	WH	98													
				8	SS	WH	97										0	1	50	49
				9	SS	WH	96													
							95													
				10	SS	WR	94													
93.6	11.3	(CL) Sandy CLAYEY SILT Firm Grey W>PL					93													
				11	SS	WH	92										1	29	47	23
92.1		(SM) Gravelly SILTY SAND, some clay (TILL) Grey Moist		12	SS	50/0.15	91													
91.7	13.2	Marble (BEDROCK)		1	RC	REC 85%	90													
		Bedrock cored from depths 13.2 m to 17.1 m.		2	RC	REC 100%														
		For bedrock coring details refer to Record of Drillhole 22-01.																		

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

GTA-MTO 001 S:\CLIENTS\MTOWHY_17_417_ARNPRIOR02_DATA\GINT\HWY_17_417_ARNPRIOR.GPJ GAL-GTA.GDT 6/4/24



PROJECT 21480555			RECORD OF BOREHOLE No 22-01				SHEET 2 OF 2			METRIC								
G.W.P. 4024-22-00			LOCATION N 5030649.6; E 318064.8 MTM NAD 83 ZONE 9 (LAT. 45.415665; LONG. -76.330521)				ORIGINATED BY KG											
DIST Eastern HWY 417			BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring				COMPILED BY TR											
DATUM Geodetic			DATE June 27, 2022				CHECKED BY KCP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100						
	Marble (BEDROCK)		2	RC	REC 100%												RQD = 97%	
	Bedrock cored from depths 13.2 m to 17.1 m.																	
	For bedrock coring details refer to Record of Drillhole 22-01.																	RQD = 98%
87.8	END OF BOREHOLE																	
17.1	NOTE: Date Depth (m) Elev. (m) 04-Oct-22 0.9 104.0 11-Oct-23 1.2 103.7 12-Dec-23 2.4 102.5																	

GTA-MTO 001 S:\CLIENTS\MTOWHY_17_417_ARNPRIOR\02_DATA\GINT\HWY_17_417_ARNPRIOR.GPJ GAL-GTA.GDT 6/4/24

PROJECT: 21480555

RECORD OF DRILLHOLE: 22-01

SHEET 1 OF 1

LOCATION: N 5030649.60 ;E 318064.78

DRILLING DATE: June 29, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 850 Track Mount

DRILLING CONTRACTOR: CCC Geotechnical & Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																	FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA					WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
		Borehole continued from Record of Borehole 22-01		91.70																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															</

DEPTH SCALE

1 : 50



LOGGED: KG

CHECKED: KCP

[illegible]

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE



PROJECT 21480555		RECORD OF BOREHOLE No 22-03		SHEET 1 OF 2		METRIC	
G.W.P. 4024-22-00		LOCATION N 5030628.4; E 318084.5 MTM NAD 83 ZONE 9 (LAT. 45.415474; LONG. -76.330269)		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem)		COMPILED BY TR			
DATUM Geodetic		DATE July 8, 2022		CHECKED BY KCP			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						
106.7	GROUND SURFACE							20 40 60 80 100						
0.7	(SM) SILTY SAND, with rootlets (TOPSOIL) Brown Moist		1	SS	42									
	(GP-GM) GRAVEL and sand, asphalt fragments (FILL) Dense Grey-brown-dark black Dry		2	SS	32									49 40 (11)
105.3														
1.4	(SM) SILTY SAND, fine to medium grained, trace clay (FILL) Compact Grey-light brown Moist		3	SS	12									
104.6														
2.1	(CH) CLAY, trace rootlets (WEATHERED CRUST) Very stiff Grey-brown w>PL		4	SS	7									
103.8														
2.9														
	START OF DCPT													
					</									

GTA-MTO 001 S:\CLIENTS\MTOWHY_17_417_ARNPRIOR\02_DATA\GINT\HWY_17_417_ARNPRIOR.GPJ GAL-GTA.GDT 6/4/24

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE



PROJECT		RECORD OF BOREHOLE				No 22-03		SHEET 2 OF 2		METRIC				
G.W.P.		4024-22-00		LOCATION		N 5030628.4; E 318084.5 MTM NAD 83 ZONE 9 (LAT. 45.415474; LONG. -76.330269)		ORIGINATED BY		DG				
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem)		COMPILED BY		TR				
DATUM		Geodetic		DATE		July 8, 2022		CHECKED BY		KCP				
SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40					
90.4	END OF BOREHOLE													
16.3	DCPT Refusal at 16.3 m													

[illegible]



PROJECT 21480555			RECORD OF BOREHOLE No 22-04			SHEET 2 OF 2			METRIC														
G.W.P. 4024-22-00			LOCATION N 5030646.3; E 318104.3 MTM NAD 83 ZONE 9 (LAT. 45.415634; LONG. -76.330016)			ORIGINATED BY KG																	
DIST Eastern HWY 417			BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring			COMPILED BY TR																	
DATUM Geodetic			DATE June 29 & 30, 2022			CHECKED BY KCP																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
--- CONTINUED FROM PREVIOUS PAGE ---																							
90.4			14	SS	8																		
15.7	(ML) gravelly sandy SILT to sandy SILT, some gravel, trace to some clay, contains cobbles (TILL)		15	SS	50/0.08																		
89.3	Dense Grey w>PL		16	SS	50/0.08																		
16.8	Marble (BEDROCK)		1	RC	REC 72%																	RQD = 0%	
	Bedrock cored from depths 16.8 m to 21.6 m.		2	RC	REC 0%																	RQD = 0%	
	For bedrock coring details refer to Record of Drillhole 22-04.		3	RC	REC 100%																	RQD = 44%	
86.3																							
19.8	Dolostone (BEDROCK)		4	RC	REC 97%																	RQD = 60%	
84.5																							
21.6	END OF BOREHOLE																						

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PROJECT: 21480555

RECORD OF DRILLHOLE: 22-04

SHEET 1 OF 1

LOCATION: N 5030646.26 ;E 318104.25

DRILLING DATE: June 30, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 850 Track Mount

DRILLING CONTRACTOR: CCC Geotechnical & Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA					WEATH- ERING INDEX			Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
							TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon	W1	W2	W3		W4			W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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17		Borehole continued from Record of Borehole 22-04		89.32 16.78		1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											

DEPTH SCALE

1 : 50



LOGGED: KG

CHECKED: KCP

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PROJECT			21480555			LOCATION			N 5030718.7; E 318161.1 MTM NAD 83 ZONE 9 (LAT. 45.416285; LONG. -76.329288)			ORIGINATED BY			DG														
G.W.P.			4024-22-00			BOREHOLE TYPE			Power Auger, 200 mm Dia. (Hollow Stem)			COMPILED BY			TR														
DIST			Eastern HWY 417			DATE			July 7, 2022			CHECKED BY			KCP														
SOIL PROFILE															SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)														
105.4	GROUND SURFACE																												
0.0	(SM) SILTY SAND, trace rootlets (TOPSOIL)		1	SS	5		105																						
104.8	Brown Moist																												
0.6	(CH) CLAY (WEATHERED CRUST)		2	SS	9		104																						
	Very stiff																												
	Grey-brown		3	SS	9		103																						
	w>PL																												
102.5			4	SS	4		102																						
2.9	START OF DCPT						101																						
							100																						
							99																						
							98																						
							97																						
							96																						
94.9							95																						
10.5	END OF BOREHOLE																												
	DCPT Refusal at 10.5 m																												

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

[illegible]



PROJECT 21480555			RECORD OF BOREHOLE No 22-06			SHEET 2 OF 2			METRIC														
G.W.P. 4024-22-00			LOCATION N 5030751.7; E 318198.0 MTM NAD 83 ZONE 9 (LAT. 45.416581; LONG. -76.328815)			ORIGINATED BY DG																	
DIST Eastern HWY 417			BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring			COMPILED BY TR																	
DATUM Geodetic			DATE July 6, 2022			CHECKED BY KCP																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
--- CONTINUED FROM PREVIOUS PAGE ---																							
88.9	(CI) SILTY CLAY Firm Grey w>PL		14	SS	WH																		
16.8	(SM) SILTY SAND (TILL) Grey Wet																						
87.3			15	SS	50/0.15																		
18.4	SANDSTONE (BEDROCK) Bedrock cored from depths 18.4 m to 21.4 m. For bedrock coring details refer to Record of Drillhole 22-06.		1	RC	REC 100%																		
84.3			2	RC	REC 100%																		
21.4	END OF BOREHOLE																						

PROJECT: 21480555

RECORD OF DRILLHOLE: 22-06

SHEET 1 OF 1

LOCATION: N 5030751.70 ;E 318198.04

DRILLING DATE: July 6, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 850 Track Mount

DRILLING CONTRACTOR: CCC Geotechnical & Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t. CORE AXIS DIP CORE	DISCONTINUITY DATA			WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KCP

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PROJECT		21480555		RECORD OF BOREHOLE		No 22-07		SHEET 1 OF 2		METRIC			
G.W.P.		4024-22-00		LOCATION		N 5030694.9; E 318182.5 MTM NAD 83 ZONE 9 (LAT. 45.416071; LONG. -76.329016)		ORIGINATED BY		KG			
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring		COMPILED BY		TR			
DATUM		Geodetic		DATE		July 5, 2022		CHECKED BY		KCP			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L			
106.3	GROUND SURFACE												
0.0	(SM/GM) Silty sand and gravel (TOPSOIL) Brown Dry		1	SS	41								
0.2	(SM/GM) Gravelly SILTY SAND (FILL) Dense to loose Brown Moist		2	SS	8								30 45 (25)
104.9	(CH) CLAY (WEATHERED CRUST) Very stiff to stiff Grey-brown w>PL		3	SS	18								
1.4			4	SS	11								
			5	SS	7								
			6	SS	5								0 0 47 53
			7	SS	3								
			8	SS	1								
99.6	(CI) SILTY CLAY Firm Grey w>PL		9	TP	-								0 1 49 50
6.7			10	TP	-								
			11	TP	-								
94.5	Dolostone (BEDROCK)		1	RC	REC 100%								RQD = 71%
11.8	Bedrock cored from depths 11.8 m to 15.3 m. For bedrock coring details refer to Record of Drillhole 22-07.		2	RC	REC 100%								RQD = 89%
			3	RC	REC 100%								RQD = 49%

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

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PROJECT <u>21480555</u>			RECORD OF BOREHOLE No 22-07				SHEET 2 OF 2			METRIC										
G.W.P. <u>4024-22-00</u>			LOCATION <u>N 5030694.9; E 318182.5 MTM NAD 83 ZONE 9 (LAT. 45.416071; LONG. -76.329016)</u>				ORIGINATED BY <u>KG</u>													
DIST <u>Eastern</u> HWY <u>417</u>			BOREHOLE TYPE <u>Power Auger, 200 mm Dia. (Hollow Stem), NQ Coring</u>				COMPILED BY <u>TR</u>													
DATUM <u>Geodetic</u>			DATE <u>July 5, 2022</u>				CHECKED BY <u>KCP</u>													
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										25 50 75		
91.0			3	RC			91										RQD = 49%			
15.3	END OF BOREHOLE																			

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PROJECT: 21480555

RECORD OF DRILLHOLE: 22-07

SHEET 1 OF 1

LOCATION: N 5030694.90 ;E 318182.47

DRILLING DATE: July 5, 2022

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850 Track Mount

DRILLING CONTRACTOR: CCC Geotechnical & Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY															FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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DEPTH SCALE

1 : 50



LOGGED: KG

CHECKED: KCP

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+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE



PROJECT 21480555				RECORD OF BOREHOLE No 22-08				SHEET 2 OF 2				METRIC						
G.W.P. 4024-22-00				LOCATION N 5030734.8; E 318215.0 MTM NAD 83 ZONE 9 (LAT. 45.416429; LONG. -76.328599)				ORIGINATED BY DG										
DIST Eastern HWY 417				BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem)				COMPILED BY TR										
DATUM Geodetic				DATE July 7, 2022				CHECKED BY KCP										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
	--- CONTINUED FROM PREVIOUS PAGE ---																	
90.0	END OF BOREHOLE						90											
15.8	DCPT refusal at 15.8 m																	

GTA-MTO 001 S:\CLIENTS\MTOWHY_17_417_ARNPRIOR\02_DATA\GINT\HWY_17_417_ARNPRIOR.GPJ GAL-GTA.GDT 6/4/24



PROJECT 21480555

G.W.P. 4024-22-00

DIST Eastern

DATUM Geodetic

LOCATION N 5030544.4; E 317965.7 MTM NAD 83 ZONE 9 (LAT. 45.414721; LONG. -76.331789)

BOREHOLE TYPE Power Auger, 200 mm Dia. (Hollow Stem)

DATE July 11, 2022

SHEET 1 OF 1

ORIGINATED BY DG

COMPILED BY TR

CHECKED BY KCP

RECORD OF BOREHOLE No SWMP1

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
104.4	GROUND SURFACE																
0.0	(SM) SILTY SAND (TOPSOIL)																
0.2	Brown Moist		1	SS	10												
	(CH) CLAY (WEATHERED CRUST)																
	Very stiff Grey-brown w>PL		2	SS	14												
			3	SS	8												
			4	SS	5												
			5	SS	4												
			6	SS	2												
100.0																	
4.4	(CH) CLAY																
	Stiff to firm Grey w>PL																
			7	SS	WH												
			8	SS	WH												
96.2																	
8.2	END OF BOREHOLE																

GTA-MTO 001 S:\CLIENTS\MTOWHY_17_417_ARNPRIOR\02_DATA\GINT\HWY_17_417_ARNPRIOR.GPJ GAL-GTA.GDT 6/4/24

[illegible]

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT		CA0012298.4565	
G.W.P.		4024-22-00	
DIST		Eastern HWY 417	
DATUM		Geodetic	
		October 16, 2023	
		KCP	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	W _P W W _L	WATER CONTENT (%)			
104.0 0.0 0.2	GROUND SURFACE TOPSOIL (CH) CLAY, trace sand, fissured (WEATHERED CRUST) Very Stiff Brownish grey, mottled w>PL		1	SS	5								
			2	SS	10								
			3	SS	8								
			4	SS	6								
			5	SS	4								
100.0 4.0	(Cl) SILTY CLAY, trace sand Stiff to firm Grey w>PL		6	SS	1								
			7	SS	1								
	- Sand seams from 7.2 m to 9.0 m		8	SS	4								
95.0 9.0 94.6 9.4	(CL-ML/ML) CLAYEY SILT-SILT to SILT of slight plasticity, some sand, some gravel (TILL) Grey w=PL Marble (BEDROCK)		9	SS	56/0.25								
	Bedrock cored from depths 9.4 m to 11.2 m.		1	RC	REC 98%								RQD = 79%
	For bedrock coring details refer to Record of Drillhole BH23-03.		2	RC	REC 100%								RQD = 76%
92.8 11.2	END OF BOREHOLE												

NOTE:

Date	Depth (m)	Elev. (m)
19-Oct-23	0.8	103.2
12-Dec-23	5.2	98.8

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: MARATHON

[illegible]

DEPTH SCALE

1 : 50



LOGGED: CR

CHECKED:

**BH 22-01 (Dry)
Core Box 1 to 2 of 2**



Note:

1. Elevation 91.2 m to 87.8 m - Marble Bedrock



**Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario**

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

Checked: KCP

Review: JPD

Figure A1

BH 22-01 (Wet)
Core Box 1 to 2 of 2



Note:

1. Elevation 91.2 m to 87.8 m - Marble Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

Checked: KCP

Review: JPD

Figure A2

BH 22-04 (Dry)
Core Box 1 to 2 of 2

Elevation 86.3 m (see Note 3)

Lost Core (see Note 2)

Elevation 89.3 m Top of Bedrock (see Note 1)



Elevation 84.5 m EOH

Note:

1. Elevation 89.3 m to 86.3 m - Marble Bedrock
2. Lost core due to mechanical malfunction between Elevation 88.6 m and 87.5 m (Run no. 2)
3. Elevation 86.3 m to 84.5 m - Dolostone Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

Checked: KCP

Review: JPD

Figure A3

BH 22-04 (Wet)
Core Box 1 to 2 of 2



Note:

1. Elevation 89.3 m to 86.3 m - Marble Bedrock
2. Lost core due to mechanical malfunction between Elevation 88.6 m and 87.5 m (Run no. 2)
3. Elevation 86.3 m to 84.5 m - Dolostone Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

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Figure A4

BH 22-06 (Dry)
Core Box 1 of 1

Elevation 87.3 m Top of Bedrock



Elevation 84.3 m EOH

Note:

1. Elevation 87.3 m to 84.3 m - Sandstone Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

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Figure A5

BH 22-06 (Wet)
Core Box 1 of 1

Elevation 87.3 m Top of Bedrock



Elevation 84.3 m EOH

Note:

1. Elevation 87.3 m to 84.3 m - Sandstone Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

Checked: KCP

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Figure A6

**BH 22-07 (Dry)
Core Box 1 to 2 of 2**

Elevation 94.5 m Top of Bedrock



Elevation 91.0 m EOH

Note:

1. Elevation 94.5 m to 91.0 m - Dolostone Bedrock



**Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario**

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

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Review: JPD

Figure A7

BH 22-07 (Wet)
Core Box 1 to 2 of 2

Elevation 94.5 m Top of Bedrock



Elevation 91.0 m EOH

Note:

1. Elevation 94.5 m to 91.0 m - Dolostone Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. 21480555-2000

Drawn: BW

Date: 2022-07-12

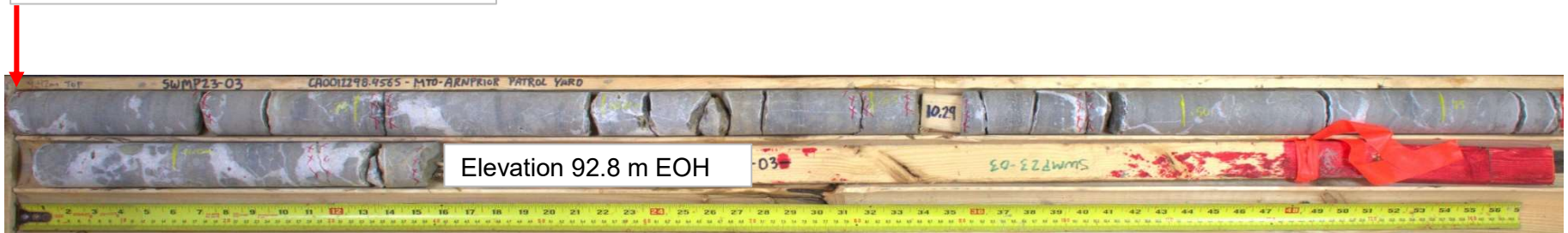
Checked: KCP

Review: JPD

Figure A8

BH 23-03 (Dry)
Core Box 1 to 1

Elevation 94.6 m Top of Bedrock



Note:

1. Elevation 94.6 m to 92.8 m - Bedrock



Foundation Investigation and Design
Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 & OR 29, Arnprior, Ontario

Project No. CA0012298.4565

Drawn: BW

Date: 2023-12-15

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Review: JPD


Figure A9

BH 23-03 (Wet)
Core Box 1 to 1

Elevation 94.6 m Top of Bedrock



Note:
1. Elevation 94.6 m to 92.8 m - Bedrock

	Foundation Investigation and Design Proposed Arnprior Maintenance Patrol Yard Location 12 - Highway 417 & OR 29, Arnprior, Ontario	Project No. CA0012298.4565	Figure A10
		Drawn: BW	
		Date: 2024-01-16	
		Checked: KCP	
		Review: JPD	

APPENDIX B

Geotechnical Laboratory Test Results

Figure B1 – Measured Engineering Properties

Figure B2 – Sandy Silt to Gravel and Sand (Fill)

Figure B3 – (CH) CLAY - Weathered Crust

Figure B4 – (CH) CLAY - Weathered Crust

Figure B5 – Plasticity Chart - Weathered Clay Crust

Figure B6 – (CI to CH) Silty Clay to Clay

Figure B7 – Plasticity Chart - Silty Clay to Clay

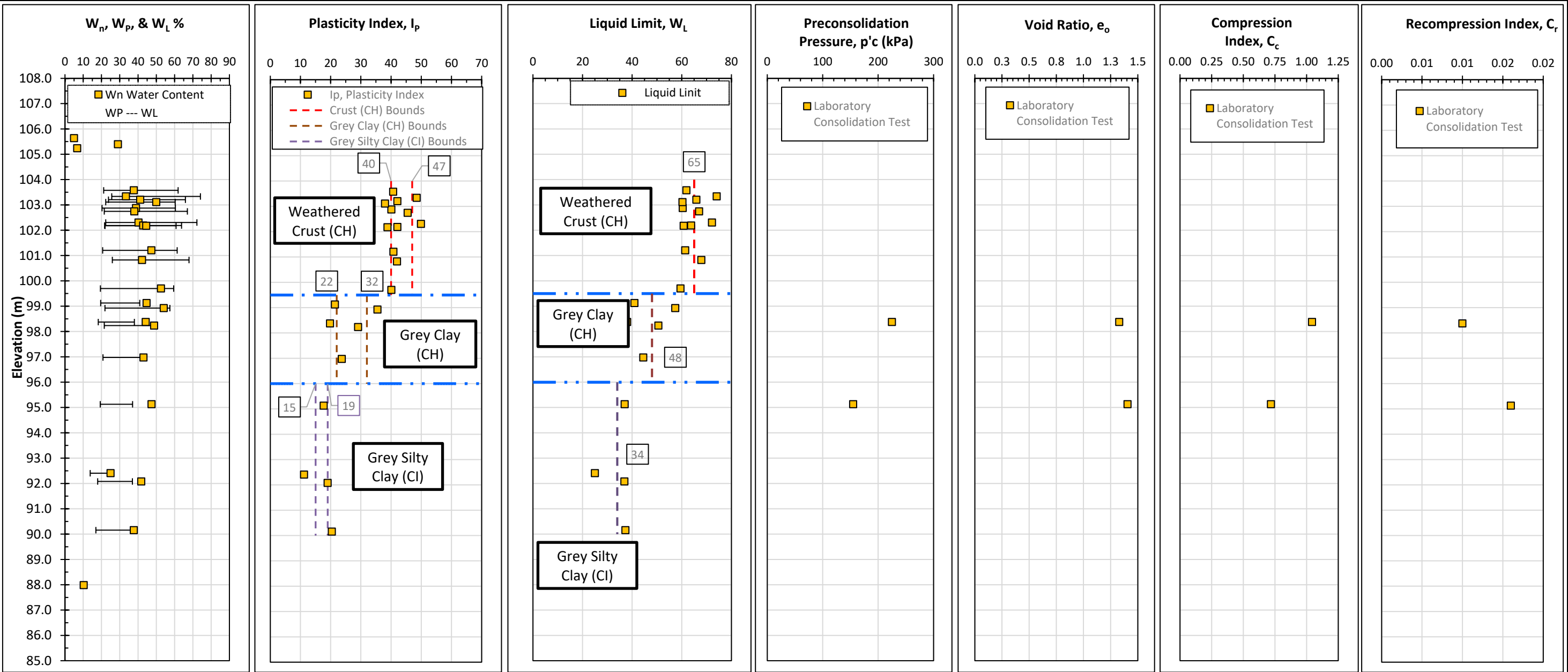
Figure B8 – (CL) Clayey Silt

Figure B9 – Plasticity Chart - (CL) Clayey Silt

Figure B10 – Plasticity Chart - (CL-ML / ML) Clayey-Silt-Silt to Silt of Slight Plasticity

Results of One-dimensional Consolidation Tests

Results of Uniaxial Compressive Strength Test of Intact Rock Core



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Engineering Properties from Laboratory Testing

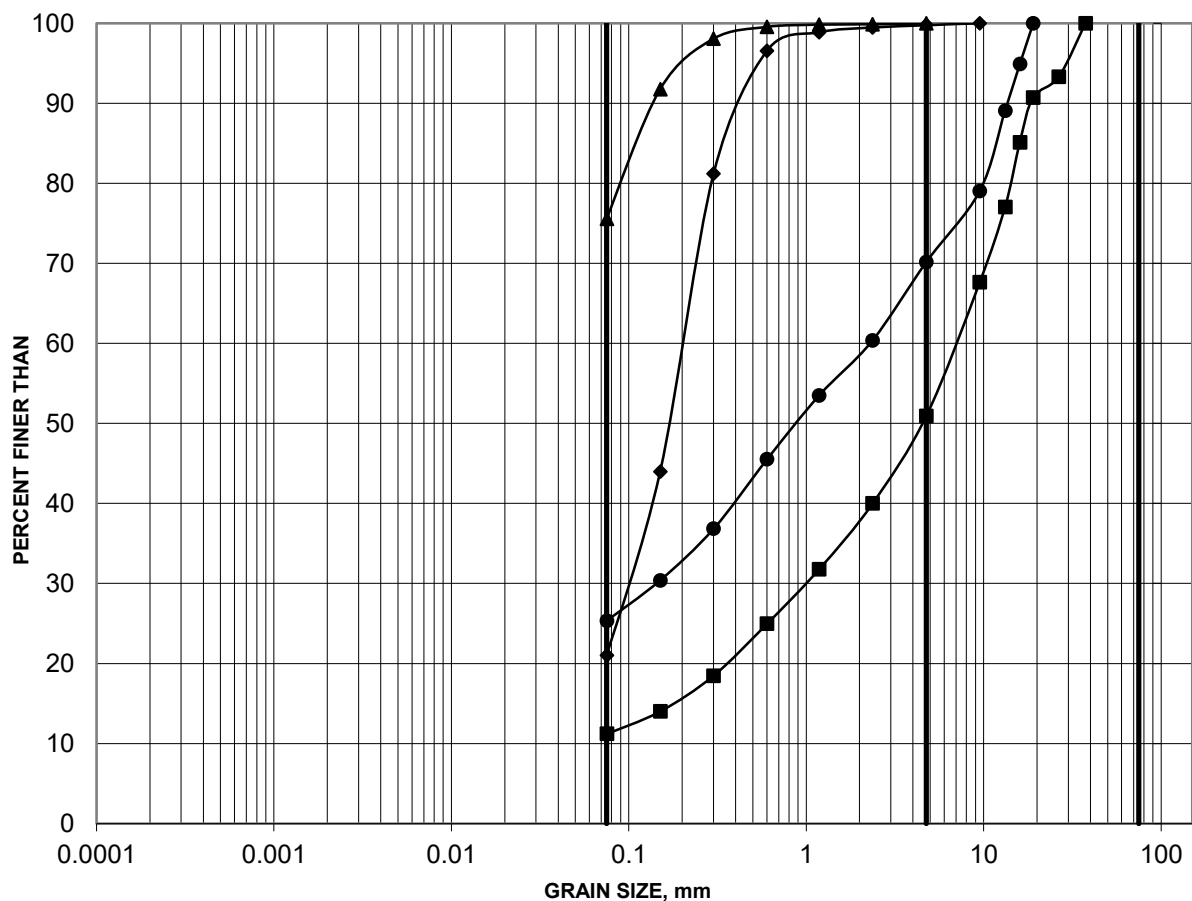
Project No.: 21480555 - 12298.4565
Date: January 2, 2024
Drawn: KCP
Review: JPD

Figure B1

GRAIN SIZE DISTRIBUTION

FIGURE B2

(ML to GP-GM) Sandy SILT to GRAVEL and sand (FILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-03	2	0.76-1.37	49	40	11	
◆	22-04	2	0.76-1.37	0	79	21	
▲	22-06	1	0.00-0.61	0	24	76	
●	22-07	2	0.76-1.37	30	45	25	

Project: 21480555/2000



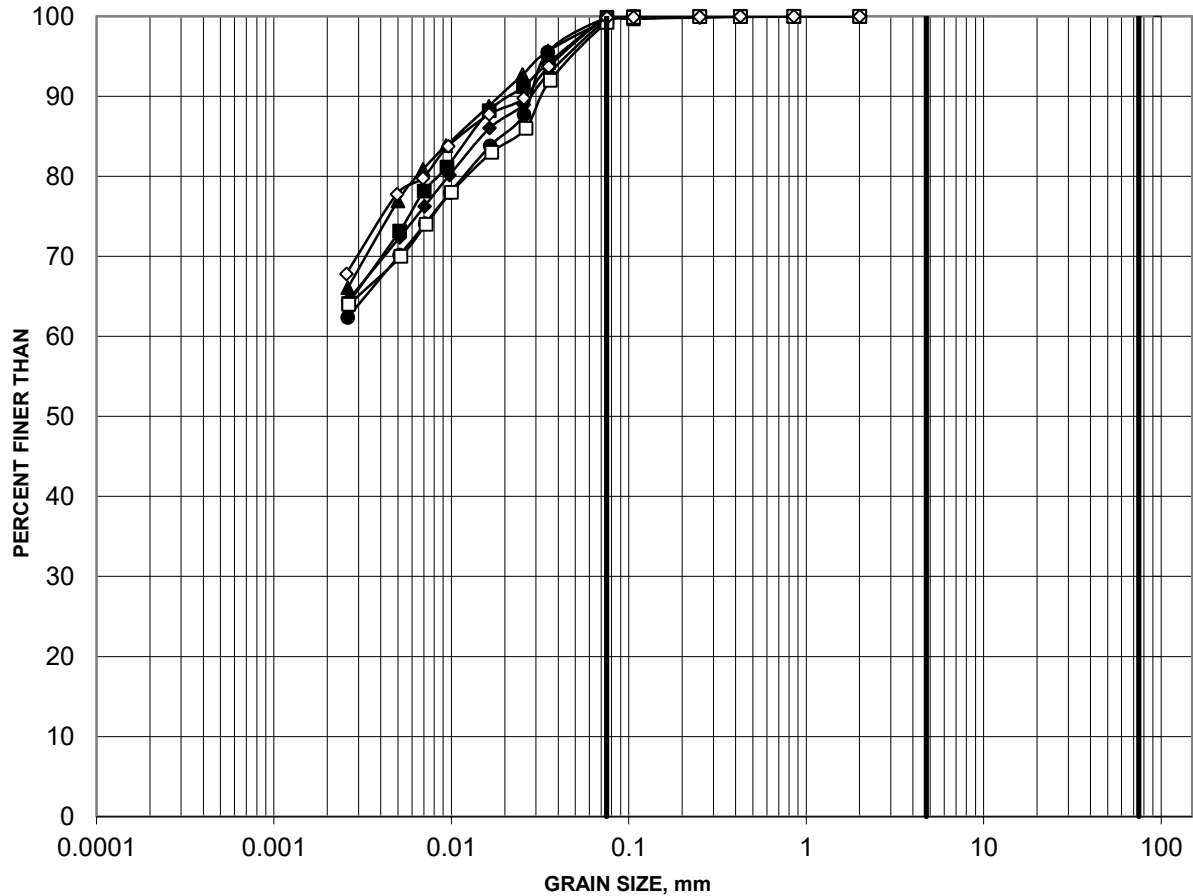
Created by: MI
Checked by: JB

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2021/21480555/Figures/>

GRAIN SIZE DISTRIBUTION

FIGURE B3

(CH) CLAY - Weathered Crust



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-01	4	2.29-2.90	0	0	41	59
◆	22-02	3	1.52-2.13	0	0	40	60
▲	22-04	5	3.05-3.66	0	1	38	61
●	22-04	8	6.10-6.71	0	0	42	58
□	22-05	3	1.52-2.13	0	1	40	59
◇	22-06	4	2.29-2.90	0	0	40	60

Project: 21480555/2000



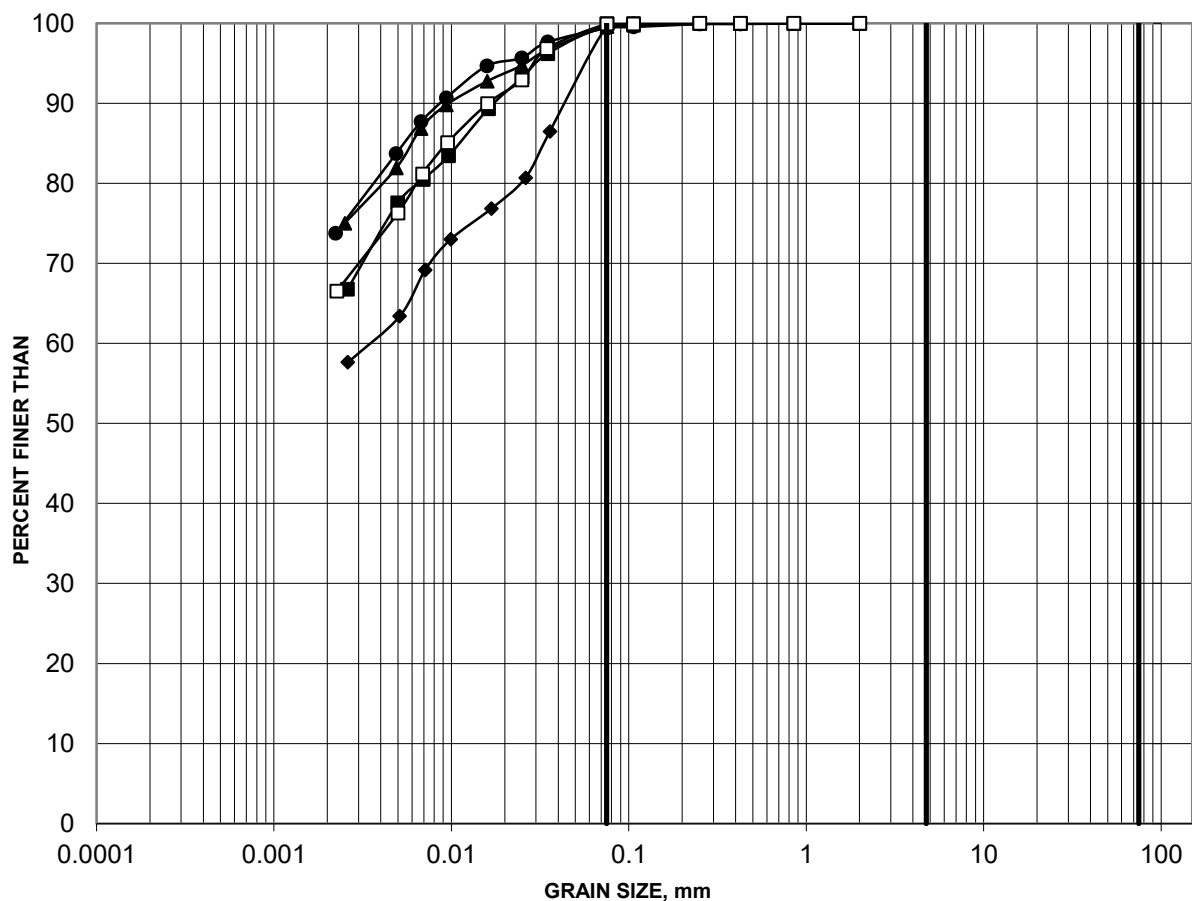
Created by: MI
Checked by: JB

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GRAIN SIZE DISTRIBUTION

FIGURE B4

(CH) CLAY - Weathered Crust



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

				Constituents (%)			
Borehole	Sample	Depth (m)		Gravel	Sand	Silt	Clay
■	22-06	7	4.57-5.18	0	0	38	62
◆	22-07	6	3.81-4.42	0	0	47	53
▲	22-08	4	2.29-2.90	0	0	30	70
●	SWMP1	2	0.76-1.37	0	1	28	71
□	SWMP2	4	2.29-2.90	0	0	36	64

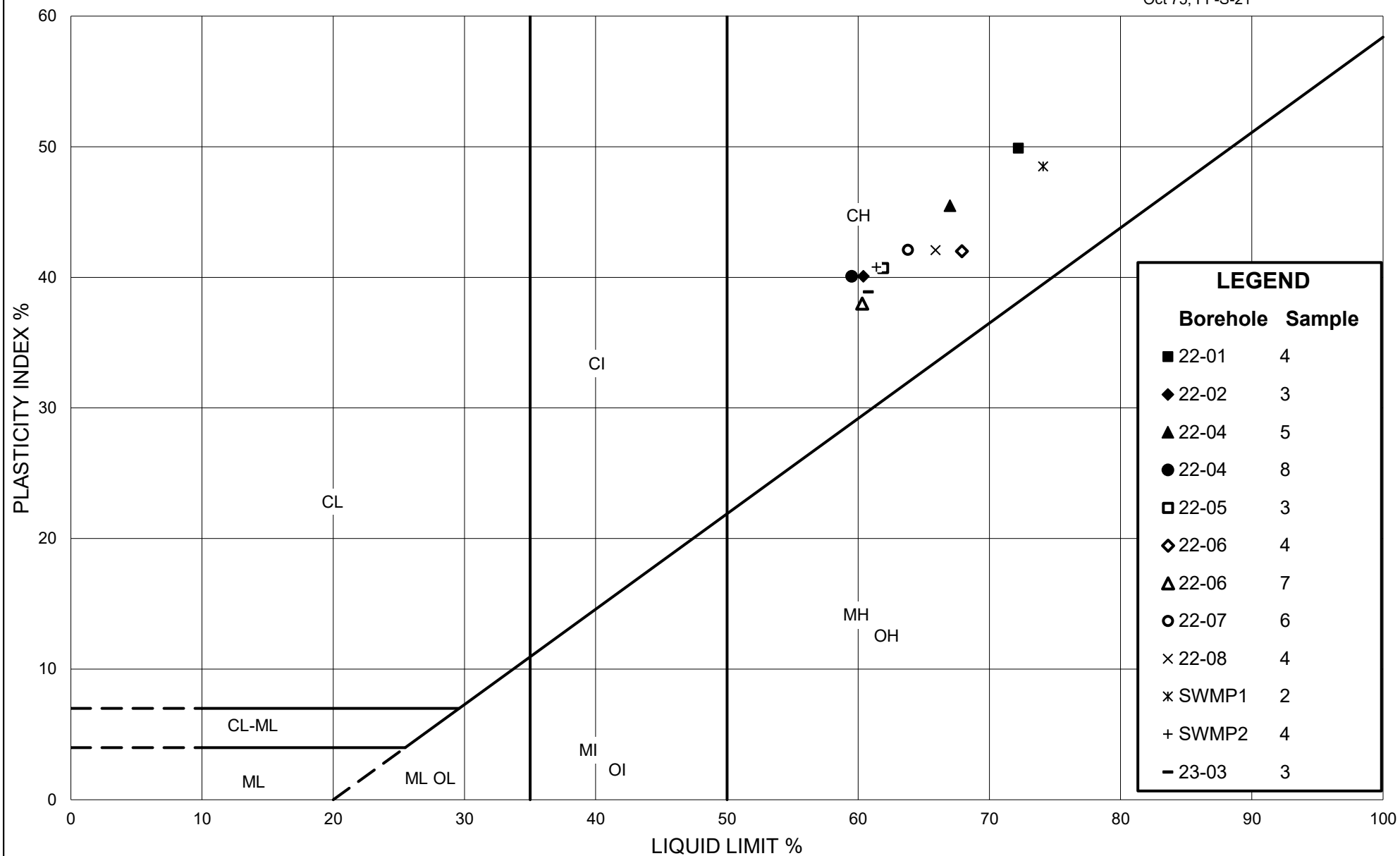
Project: 21480555/2000



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Ministry of Transportation

PLASTICITY CHART WEATHERED CLAY CRUST

Figure: B

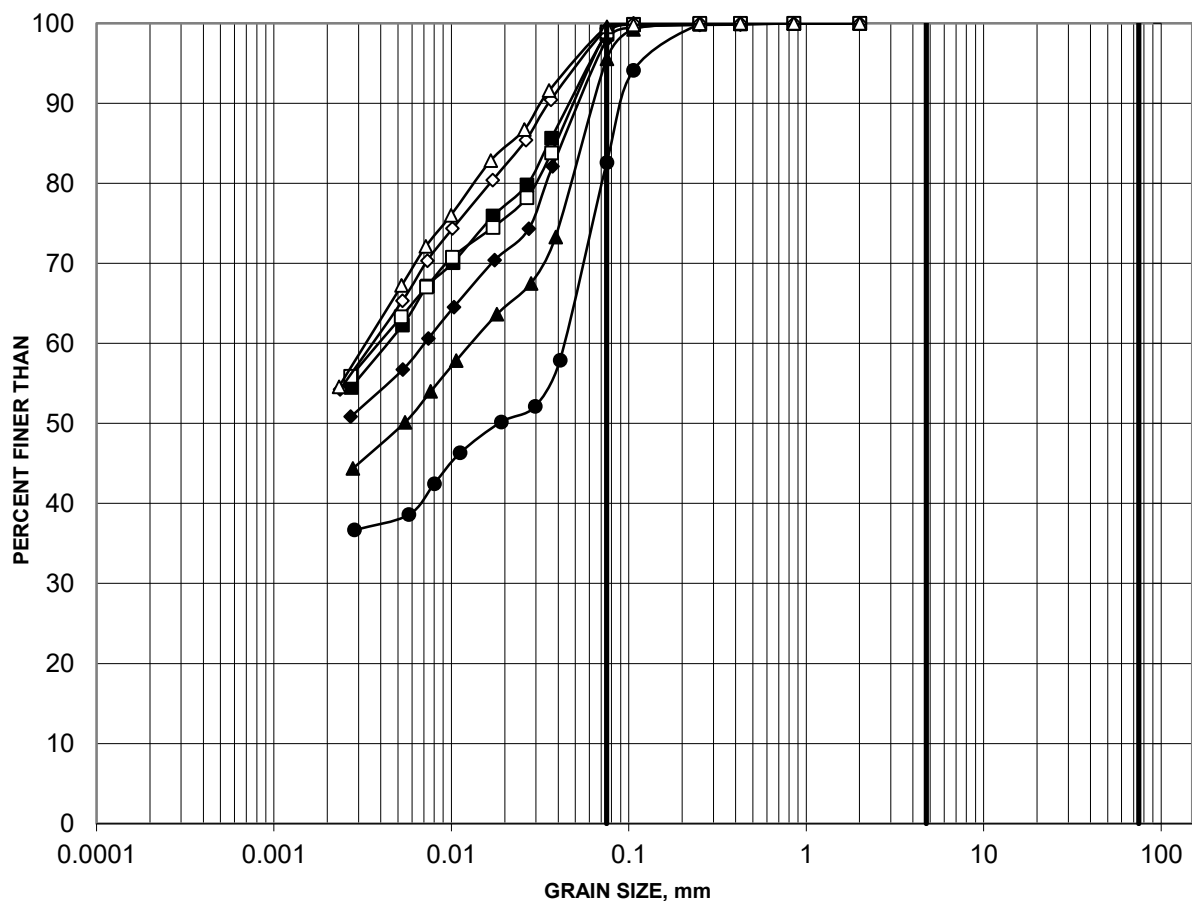
Project: 21480555/2000 / 12298.4565

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GRAIN SIZE DISTRIBUTION

FIGURE B6

(CI to CH) SILTY CLAY TO CLAY



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

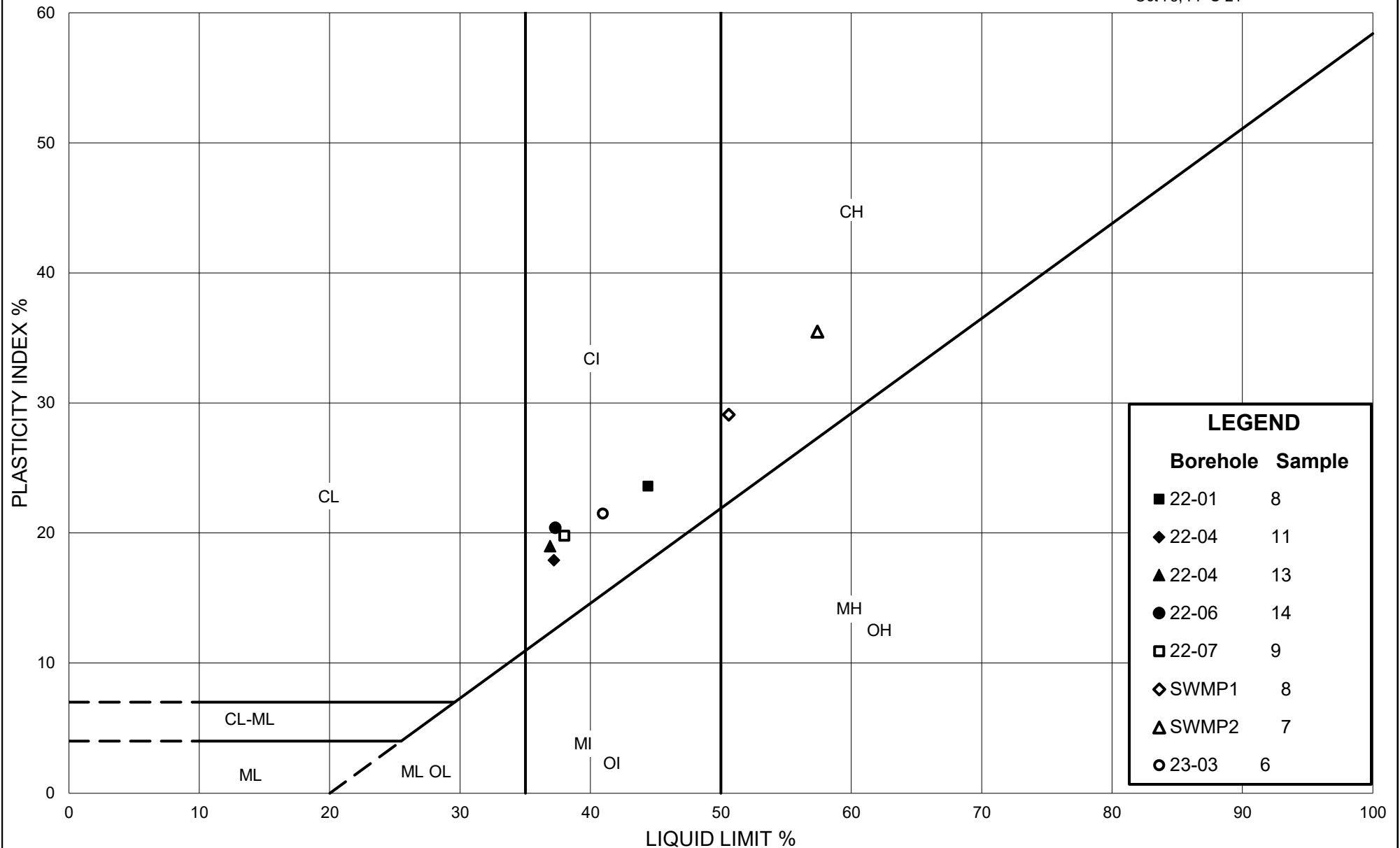
	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	22-01	8	7.62-8.23	0	1	50	49
◆	22-04	11	10.67-11.28	0	2	52	46
▲	22-04	13	13.72-14.33	0	4	55	41
●	22-06	14	15.24-15.85	0	17	50	33
□	22-07	9	7.62-8.23	0	1	49	50
◇	SWMP1	8	6.86-7.47	0	1	48	51
△	SWMP2	7	4.57-5.18	0	0	49	51

Project: 21480555/2000



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Ministry of Transportation

PLASTICITY CHART

SILTY CLAY TO CLAY

Figure: B

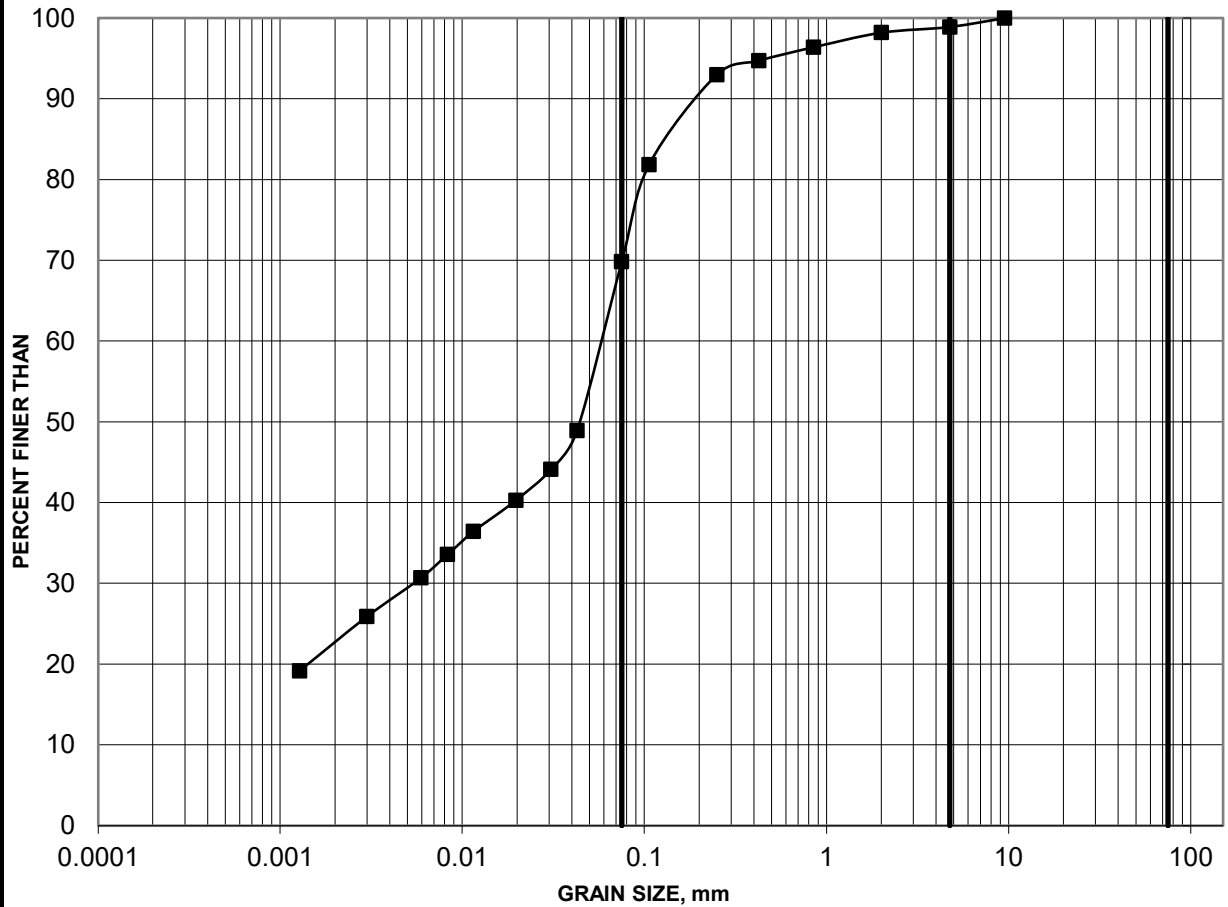
Project: 21480555/2000 / 12298.4565

Created By: MI / KCP Checked By: JB / MI

GRAIN SIZE DISTRIBUTION

FIGURE B9

(CL) CLAYEY SILT



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

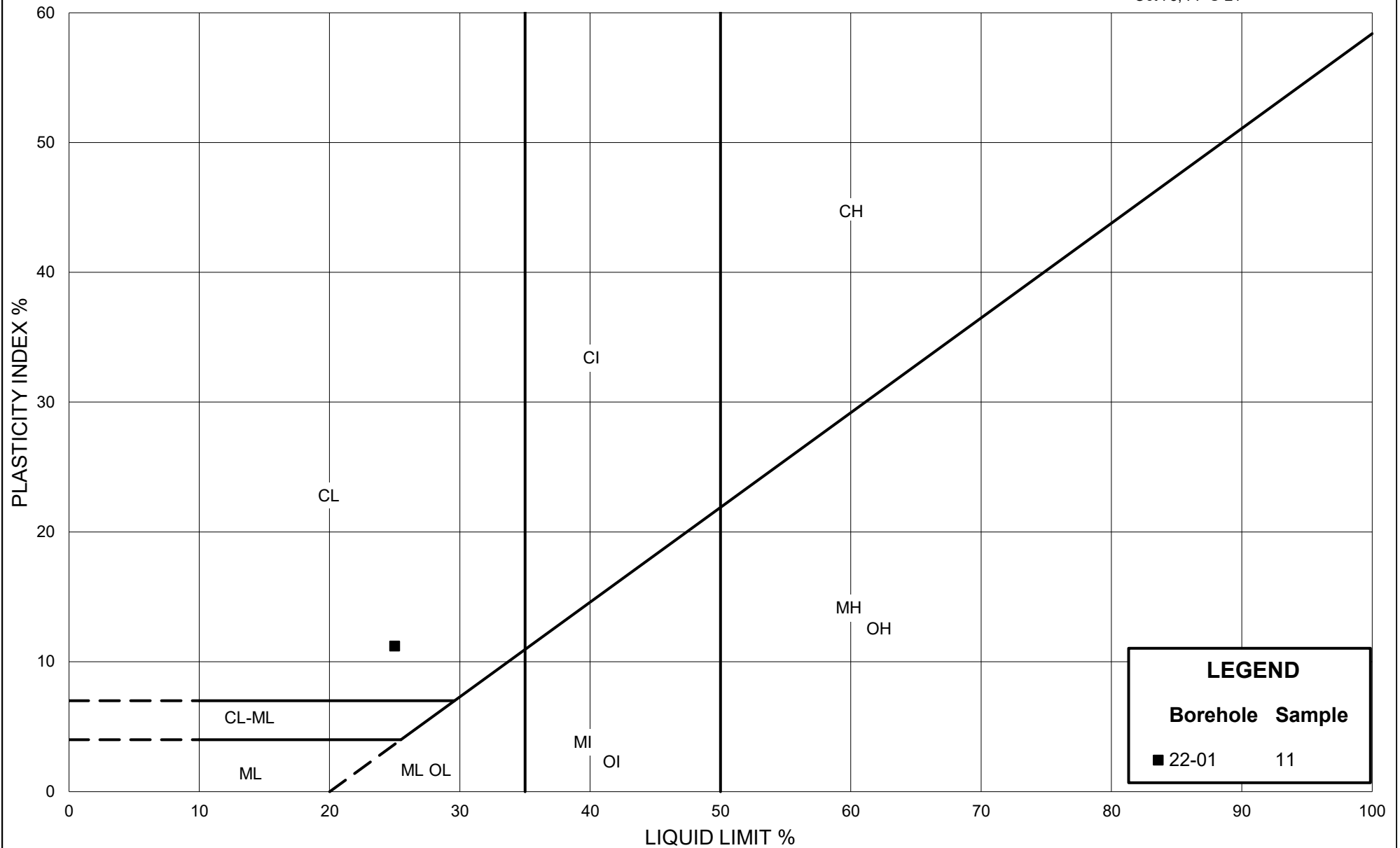
Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 21-01	11	12.19-12.80	1	29	47	23

Project: 21480555/2000



Created by: MI

Checked by: JB



Ministry of Transportation

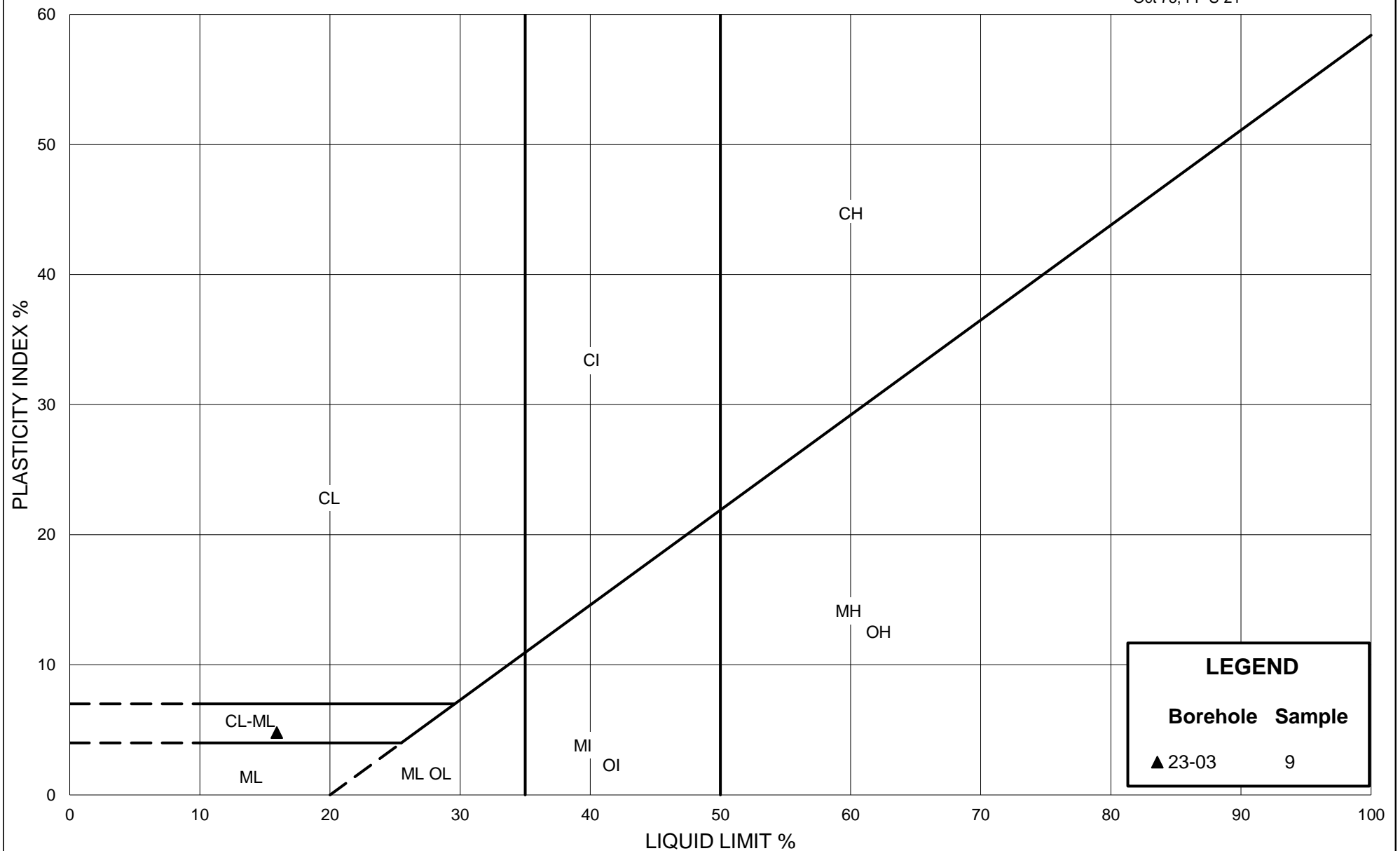
PLASTICITY CHART (CL) CLAYEY SILT

Figure: B

Project: 21480555/2000

Created By: MI

Checked By: JB



Ministry of Transportation

Ontario

PLASTICITY CHART

(CL-ML / ML) Clayey-Silt-Silt to Silt of Slight Plasticity

Figure: B 0

Project: CA0012298.4565

Created By: KCP Checked By: MI



Stantec Consulting Ltd.
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4

August 4, 2022
File: 121623407

Attention: Kenton Power, P.Eng., MASc
Wsp GOLDER
1931 Robertson Road
Ottawa, Ontario, Canada, K2H 5B7
Tel: 1-613-592-9600
E-mail: kpower@golder.com

Dear Mr. Power,

**Reference: Consolidation Test Results: Arnprior MPY, Golder, Member of WSP,
File # 21480555-2000**

This letter presents the results of one-dimensional consolidation test carried out on two shelly tubes samples in accordance with ASTM D2435/D2435M – 11(2020). The test results are provided in the attached tables and figures.

Summary of samples tested

Sample ID	Depth (ft)	Date sampled
BH 22-04 ST-11	35-37	June 29, 2022
BH 22-07 ST-09	25-27	July 5, 2022

This letter provides test results only and does not constitute any interpretation or engineering recommendations with respect to material suitability or specification compliance.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Regards,

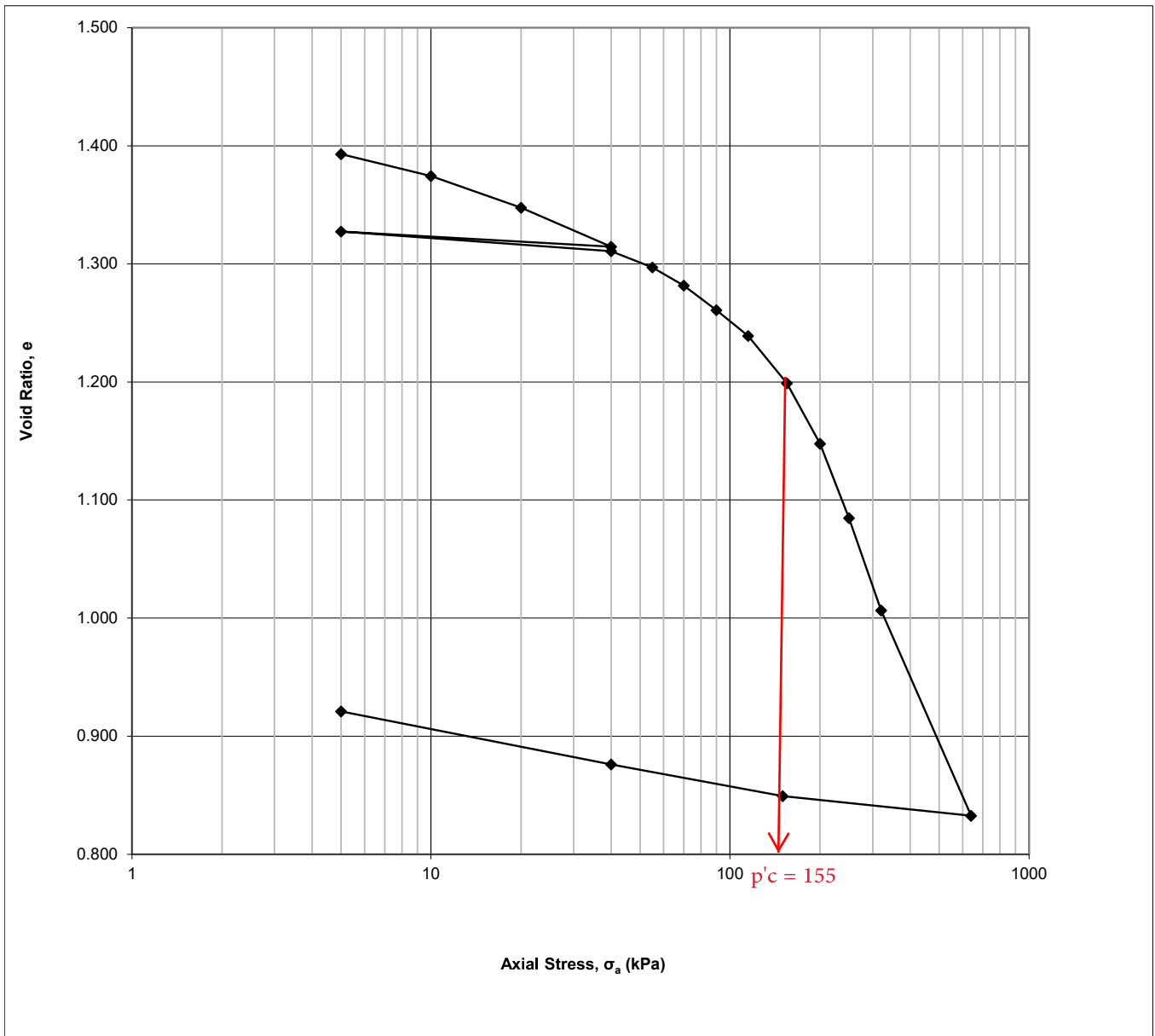
Stantec Consulting Ltd.

Ramin Ghassemi Ph.D., P.Eng.
Geotechnical Engineer
Direct: 613 722-4420
Mobile: 437 775-7625
Ramin.ghassemi@stantec.com

v:\01216\active\laboratory_standing_offers\2022-laboratory standing offers\121623407 golder associates\soils\2 consols, 5 mc., 7 hydros, limits, 2 sg, 3 ucs, file#21480555-2000\121623407_let Consolidation_bh 22-04 st 11 & 22-07 st 9.docx

Project
Project No.
Borehole No.
Sample No.
Sample Depth

wsp Golder, File# 21480555-2000
121623407
BH-04
ST 11
35-37 ft





Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	wsp Golder, File# 21480555-2000
Project Location	Arnprior, ON
Borehole	BH-04
Sample No.	ST 11
Depth	35-37 ft
Sample Date	June 29, 2022
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

<i>Lean clay, grey, wet-Cl</i>	
Specific Gravity of Solids	2.758
Liquid Limit %	37.2
Plastic Limit %	19.3
Plasticity Index %	17.9
Average water content of trimmings %	47.37
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
1. Sample flows with minimal disturbance (extremely sensitive) 2. Consolidation specimen taken @ 36'6" - 36'7"	
3. Loading schedule was provided by the Client	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	67.73
Dry Mass	g	45.01
Density	Mg/m ³	1.725
Dry Density	Mg/m ³	1.146
Water Content	%	50.48
Degree of Saturation	%	99.0
Height of Solids	mm	8.31
Initial Void Ratio		1.406

Final Specimen Conditions

Water Content	%	35.30
Final Void Ratio		0.921
Final Height	mm	15.97

One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	wsp Golder, File# 21480555-2000
Project Location	Arnprior, ON
Borehole	BH-04
Sample No.	ST 11
Depth	35-37 ft
Sample Date	June 29, 2022
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started	July 26, 2022
Date Finished	July 27, 2022
Machine Number	Frame C
Cell Number	C
Ring Number	C
Trimming Procedure	Trimming turntable/Cutting ring
Moisture Condition	Inundated
Axial Stress at Inundation kPa	5
Water Used	Deaired tap water
Test Method	B
Interpretation Procedure for c_v	2

All Departures from Outlined ASTM D2435/D2435M-11 (2020) Procedure

--

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ϵ_a %	Void Ratio e
Seating	0.0	0	0.0000	20.0000	0.00	1.406
1	30.0	5	0.1007	19.8993	0.55	1.393
2	33.3	10	0.2439	19.7561	1.32	1.374
3	40.0	20	0.4535	19.5465	2.44	1.348
4	45.0	40	0.7071	19.2929	3.81	1.315
5	30.0	5	0.6543	19.3457	3.28	1.327
6	30.0	40	0.7781	19.2219	3.97	1.311
7	45.0	55	0.8743	19.1257	4.54	1.297
8	55.3	70	1.0007	18.9993	5.18	1.282
9	68.5	90	1.1544	18.8456	6.04	1.261
10	67.0	115	1.3294	18.6706	6.95	1.239
11	97.5	155	1.6123	18.3877	8.62	1.199
12	151.8	200	2.0187	17.9813	10.75	1.148
13	171.0	250	2.5978	17.4022	13.37	1.085
14	206.5	320	3.1447	16.8553	16.61	1.007
15	133.0	640	4.4535	15.5465	23.84	0.833
16	30.0	150	4.6267	15.3733	23.14	0.849
17	45.3	40	4.4052	15.5948	22.03	0.876
18	91.0	5	4.0449	15.9551	20.17	0.921



Stantec Consulting Ltd.

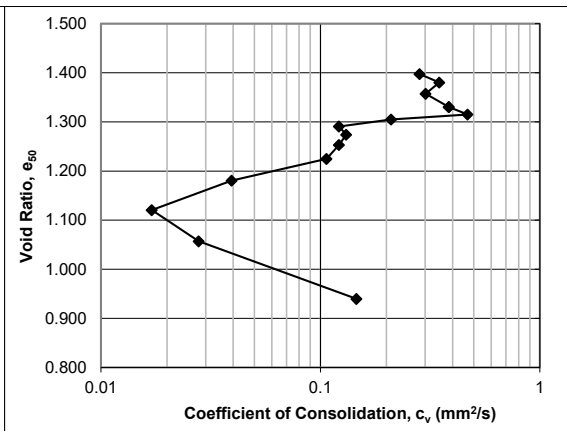
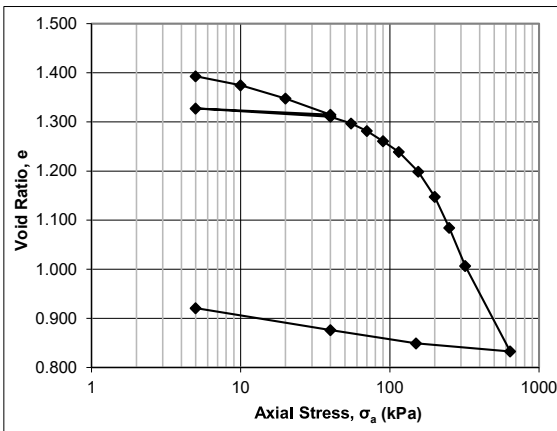
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	wsp Golder, File# 21480555-2000
Job Location	Arnprior, ON
Borehole	BH-04
Sample No.	ST 11
Depth	35-37 ft
Sample Date	June 29, 2022
Test Number	One
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm ² /s	Time t_{90} sec	Coeff. Consol. c_v mm ² /s
Seating	0								
1	3	0.0737	19.9263	0.37	1.397			297	2.83E-01
2	8	0.2163	19.7837	1.08	1.380			238	3.49E-01
3	15	0.4058	19.5942	2.03	1.357			270	3.01E-01
4	30	0.6323	19.3677	3.16	1.330			207	3.84E-01
5	23	0.6802	19.3198	3.40	1.324			680	1.16E-01
6	23	0.7556	19.2444	3.78	1.315				
7	48	0.8403	19.1597	4.20	1.305			370	2.10E-01
8	63	0.9583	19.0417	4.79	1.291			633	1.21E-01
9	80	1.0990	18.9010	5.50	1.274			578	1.31E-01
10	103	1.2742	18.7258	6.37	1.253			612	1.21E-01
11	135	1.5085	18.4915	7.54	1.225			680	1.07E-01
12	178	1.8756	18.1244	9.38	1.181			1766	3.94E-02
13	225	2.3739	17.6261	11.87	1.121			3867	1.70E-02
14	285	2.9033	17.0967	14.52	1.057			2223	2.79E-02
15	480	3.8778	16.1222	19.39	0.940			378	1.46E-01
16	395	4.6626	15.3374	23.31	0.845				
17	95	4.5023	15.4977	22.51	0.865				
18	23	4.2186	15.7814	21.09	0.899				





Project No.: 121623407

Project Name: wsp Golder, File# 21480555-2000

Photo Log

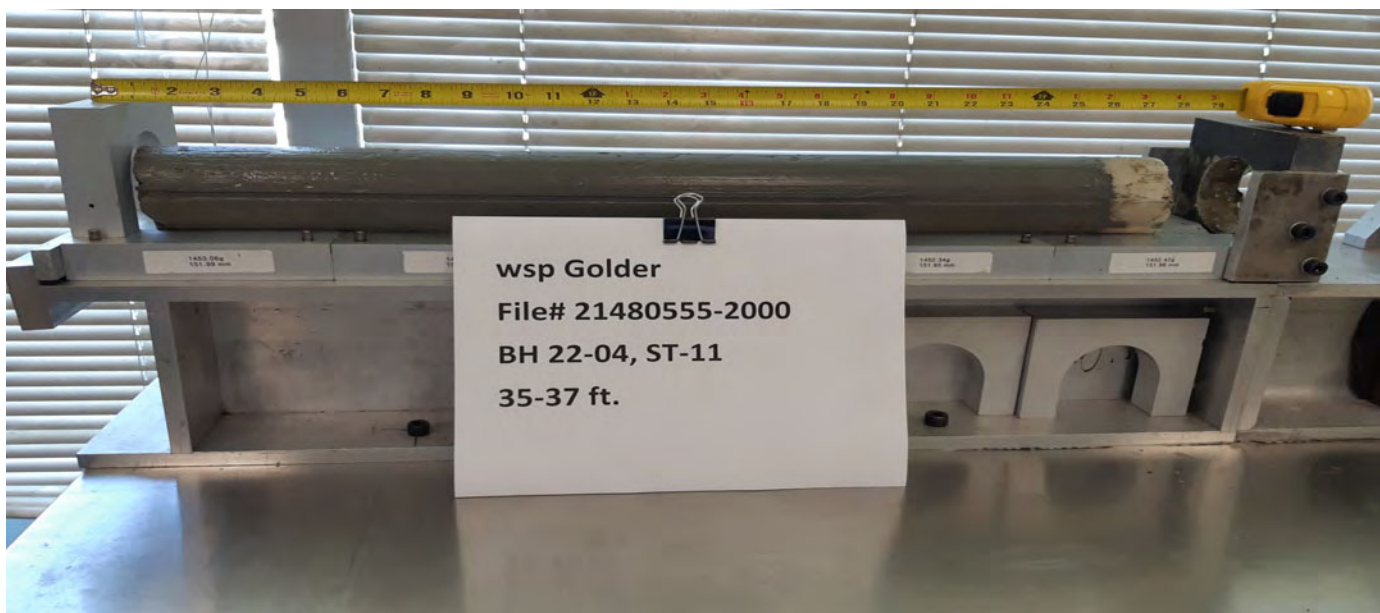


Photo No.:

1

Borehole: BH 22-04 ST-11

Depth: 35 – 37 ft

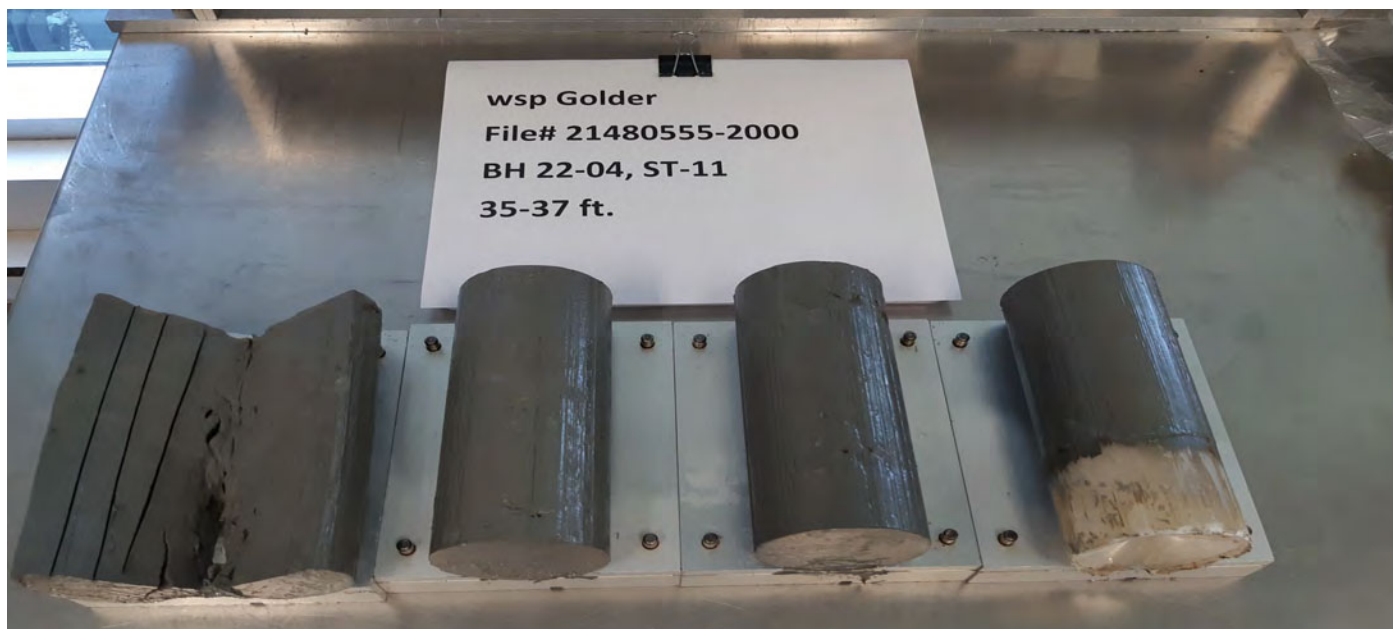


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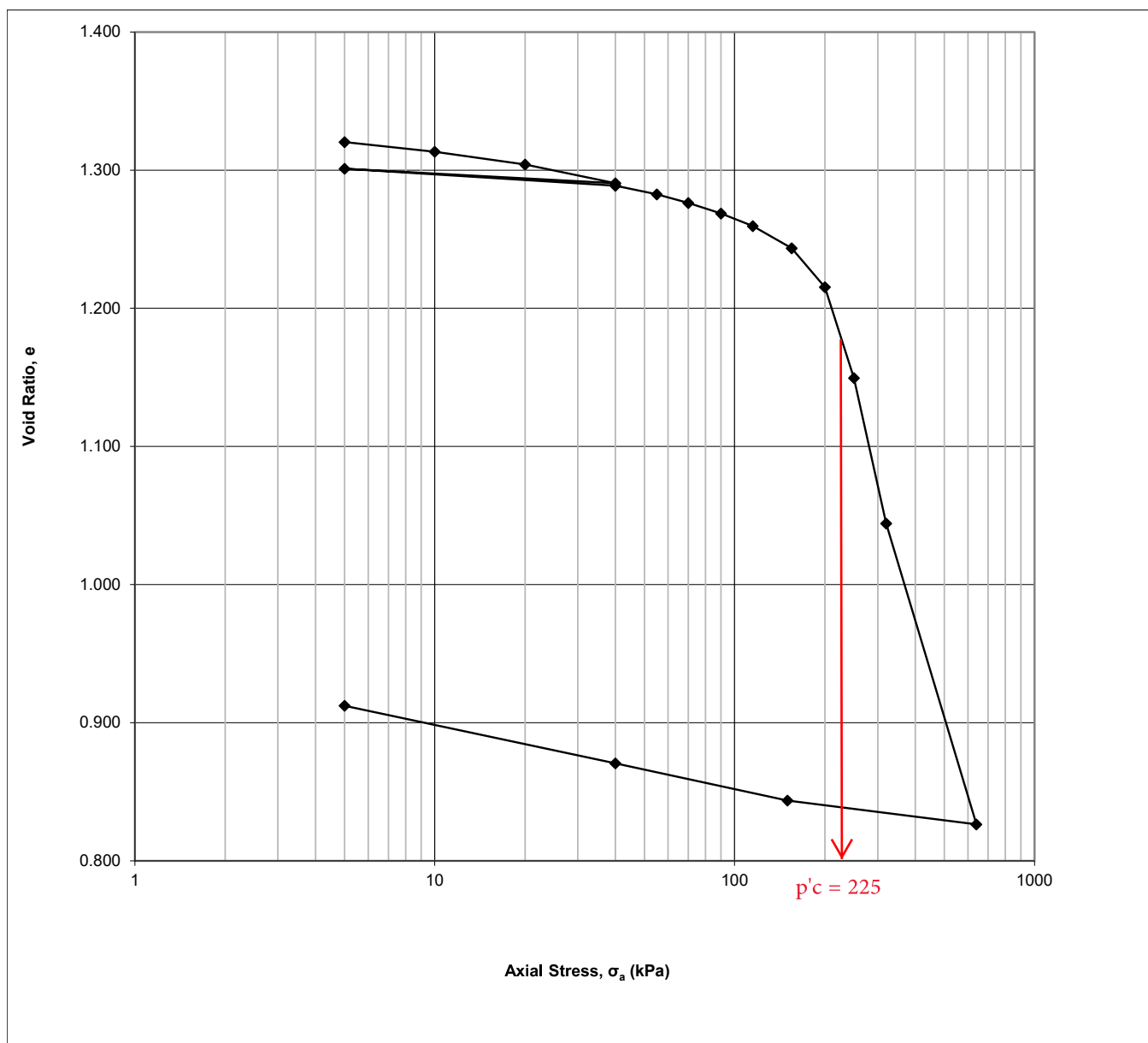
2

Borehole: BH 22-04 ST-11

Depth: 35 – 37 ft

Project
Project No.
Borehole No.
Sample No.
Sample Depth

wsp Golder, File# 21480555-2000
121623407
BH-07
ST 9
25-27 ft





Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	wsp Golder, File# 21480555-2000
Project Location	Arnprior, ON
Borehole	BH-07
Sample No.	ST 9
Depth	25-27 ft
Sample Date	July 5, 2022
Test Number	Two
Technician Name	Daniel Boateng

Soil Description & Classification

<i>Lean clay, brown/grey, friable, very moist-CI</i>	
Specific Gravity of Solids	2.747
Liquid Limit %	38.0
Plastic Limit %	18.2
Plasticity Index %	19.8
Average water content of trimmings %	44.20
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
1. Sample flows with some disturbance (sensitive) 2. Consolidation specimen taken @ 25'6" - 25'7"	
3. Loading schedule was provided by the Client	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	68.45
Dry Mass	g	46.32
Density	Mg/m ³	1.743
Dry Density	Mg/m ³	1.180
Water Content	%	47.78
Degree of Saturation	%	98.8
Height of Solids	mm	8.59
Initial Void Ratio		1.329

Final Specimen Conditions

Water Content	%	34.84
Final Void Ratio		0.912
Final Height	mm	16.42

One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	wsp Golder, File# 21480555-2000
Project Location	Arnprior, ON
Borehole	BH-07
Sample No.	ST 9
Depth	25-27 ft
Sample Date	July 5, 2022
Test Number	Two
Technician Name	Daniel Boateng

Test Procedure

Date Started	July 26, 2022
Date Finished	July 27, 2022
Machine Number	Frame D
Cell Number	D
Ring Number	D
Trimming Procedure	Trimming turntable/Cutting ring
Moisture Condition	Inundated
Axial Stress at Inundation	5 kPa
Water Used	Deaired tap water
Test Method	B
Interpretation Procedure for c_v	2

All Departures from Outlined ASTM D2435/D2435M-11 (2020) Procedure
Calculations

Load	Increment	Axial	Corrected	Specimen	Axial	Void
Increment	Duration	Stress	Deformation	Height	Strain	Ratio
	min	σ_a kPa	ΔH mm	H mm	ϵ_a %	e
Seating	0.0	0	0.0000	20.0000	0.00	1.329
1	30.0	5	0.0642	19.9358	0.37	1.320
2	30.0	10	0.1233	19.8767	0.67	1.313
3	30.0	20	0.2018	19.7982	1.06	1.304
4	30.0	40	0.3163	19.6837	1.65	1.291
5	30.0	5	0.2390	19.7610	1.20	1.301
6	30.0	40	0.3365	19.6635	1.72	1.289
7	30.0	55	0.3873	19.6127	1.99	1.283
8	30.0	70	0.4417	19.5583	2.26	1.276
9	33.3	90	0.4972	19.5028	2.59	1.269
10	36.8	115	0.5763	19.4237	2.98	1.260
11	48.5	155	0.6877	19.3123	3.67	1.243
12	97.3	200	0.8731	19.1269	4.87	1.215
13	226.3	250	1.3439	18.6561	7.70	1.149
14	250.0	320	2.3042	17.6958	12.22	1.044
15	142.8	640	3.9133	16.0867	21.58	0.826
16	30.0	150	4.1651	15.8349	20.84	0.844
17	43.3	40	3.9366	16.0634	19.68	0.871
18	76.5	5	3.5865	16.4135	17.89	0.912



Stantec Consulting Ltd.

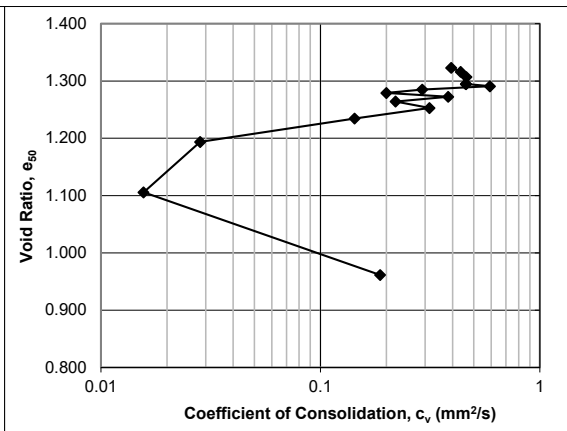
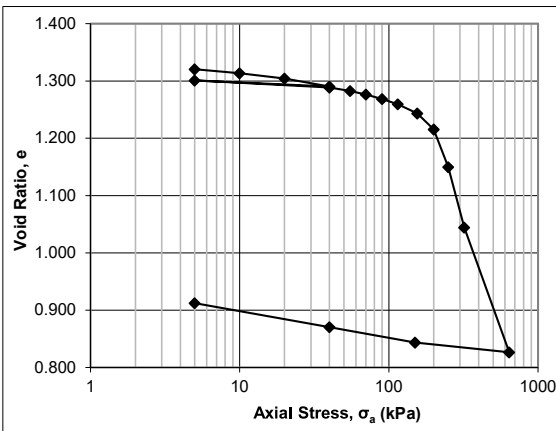
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	wsp Golder, File# 21480555-2000
Job Location	Arnprior, ON
Borehole	BH-07
Sample No.	ST 9
Depth	25-27 ft
Sample Date	July 5, 2022
Test Number	Two
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm ² /s	Time t_{90} sec	Coeff. Consol. c_v mm ² /s
Seating	0								
1	3	0.0520	19.9480	0.26	1.323			214	3.94E-01
2	8	0.1111	19.8889	0.56	1.316			192	4.36E-01
3	15	0.1872	19.8128	0.94	1.307			180	4.63E-01
4	30	0.2948	19.7052	1.47	1.295			178	4.61E-01
5	23	0.2555	19.7445	1.28	1.299				
6	23	0.3277	19.6723	1.64	1.291			138	5.93E-01
7	48	0.3757	19.6243	1.88	1.285			280	2.91E-01
8	63	0.4274	19.5726	2.14	1.279			407	2.00E-01
9	80	0.4825	19.5175	2.41	1.273			211	3.83E-01
10	103	0.5562	19.4438	2.78	1.264			364	2.20E-01
11	135	0.6539	19.3461	3.27	1.253			252	3.14E-01
12	178	0.8102	19.1898	4.05	1.235			545	1.43E-01
13	225	1.1609	18.8391	5.80	1.194			2655	2.83E-02
14	285	1.9177	18.0823	9.59	1.106			4431	1.56E-02
15	480	3.1558	16.8442	15.78	0.961			321	1.87E-01
16	395	4.2038	15.7962	21.02	0.839				
17	95	4.0372	15.9628	20.19	0.859				
18	23	3.7542	16.2458	18.77	0.892				





Project No.: 121623407

Project Name: wsp Golder, File# 21480555-2000

Photo Log

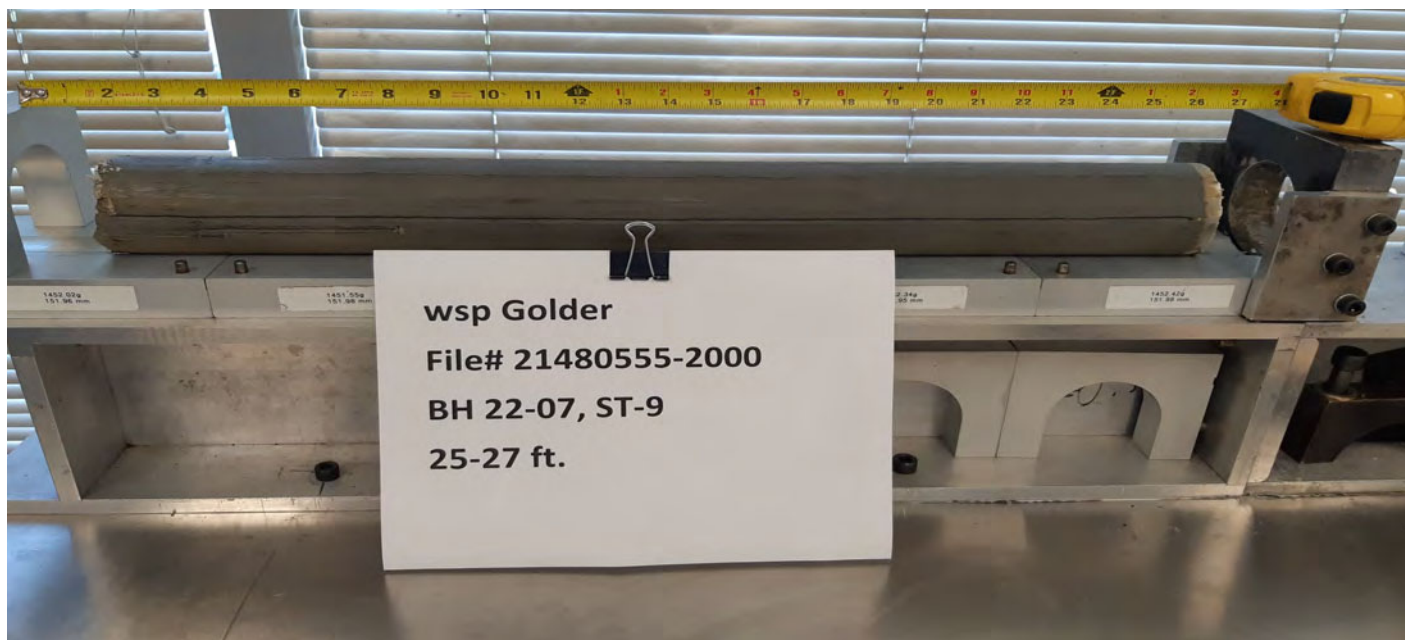


Photo No.:

1

Borehole: BH 22-07 ST-9

Depth: 25 – 27 ft



Photo No.:

2

Borehole: BH 22-07 ST-9

Depth: 25 – 27 ft



Stantec Consulting Ltd.
2781 Lancaster Rd, Suite 100 A&B, Ottawa ON K1B 1A7

August 9, 2022
File: 121623407

Client: WSP-Golder, File #21480555-2000

Reference: ASTM D7012, Method C, Unconfined Compressive Strength of Intact Rock Core, Arnprior MPY Project

The following table summarizes unconfined compressive strength results for three intact rock cores.

Location	Sample Depth (m)	Compressive Strength (MPa)	Description of Break
BH 22-07	12.40 - 13.25	185.8	Well-formed cone
BH 22-06	18.87 - 19.80	26.0	Well-formed cone
BH 22-01	13.91 - 14.28	206.2	End to end fracture

Sincerely,

Stantec Consulting Ltd.

Brian Prevost
Laboratory Supervisor
Tel: 613-738-6075
Fax: 613-722-2799
brian.prevost@stantec.com

APPENDIX C

Results of Chemical Analysis

Eurofins Environmental Testing Report Number 1987297 & 1982044

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario
K2H 5B7
Attention: Mr. Kenton Power
PO#:
Invoice to: Golder Associates Ltd

Report Number: 1982044
Date Submitted: 2022-07-20
Date Reported: 2022-07-27
Project: 21480555
COC #: 893738

Page 1 of 3

Dear Kenton Power:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL:

Sarah Horner, Inorganics Technician

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: <https://directory.cala.ca/>.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario
K2H 5B7
Attention: Mr. Kenton Power
PO#:
Invoice to: Golder Associates Ltd

Report Number: 1982044
Date Submitted: 2022-07-20
Date Reported: 2022-07-27
Project: 21480555
COC #: 893738

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	
Group	Analyte	MRL	Units	Guideline	1639006 Soil 2022-07-05 22-07 Sa 7 / 15-17'	1639007 Soil 2022-07-06 22-06 Sa 5 / 10-12'
Anions	Cl	0.002	%		0.003	0.003
	SO4	0.01	%		0.04	0.06
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.17	0.13
	pH	2.00			8.14	8.29
	Resistivity	1	ohm-cm		5880	7690

Guideline =

* = Guideline Exceedence

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario
K2H 5B7
Attention: Mr. Kenton Power
PO#:
Invoice to: Golder Associates Ltd

Report Number: 1982044
Date Submitted: 2022-07-20
Date Reported: 2022-07-27
Project: 21480555
COC #: 893738

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 426257 Analysis/Extraction Date 2022-07-26 Analyst IP Method AG SOIL			
SO4	<0.01 %	97	70-130
Run No 426367 Analysis/Extraction Date 2022-07-27 Analyst AsA Method C CSA A23.2-4B			
Chloride	<0.002 %		90-110
Run No 426369 Analysis/Extraction Date 2022-07-27 Analyst IP Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	6.14	101	90-110
Resistivity			

Guideline = * = **Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: Golder Associates Ltd

Report Number: 1987297
Date Submitted: 2022-10-04
Date Reported: 2022-10-12
Project:
COC #: 900931

Page 1 of 3

Dear Kenton Power:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

APPROVAL:

Emma-Dawn Ferguson, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: <https://directory.cala.ca/>.

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Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: Golder Associates Ltd

Report Number: 1987297
Date Submitted: 2022-10-04
Date Reported: 2022-10-12
Project:
COC #: 900931

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.
					1654424 Water 2022-10-04 BH22-01
Group	Analyte	MRL	Units	Guideline	
Anions	Cl	1	mg/L		13
	SO4	1	mg/L		62
General Chemistry	Conductivity	5	uS/cm		425
	pH	1.00			8.26
	Resistivity	0.2	Mohm-cm		<0.2

Guideline =

* = Guideline Exceedence

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: Golder Associates Ltd (Ottawa)
1931 Robertson Road,
Ottawa, Ontario

Attention: Mr. Kenton Power

PO#:

Invoice to: Golder Associates Ltd

Report Number: 1987297
Date Submitted: 2022-10-04
Date Reported: 2022-10-12
Project:
COC #: 900931

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 430930 Analysis/Extraction Date 2022-10-06 Analyst AaN Method SM 4110			
Chloride	<1 mg/L	100	90-110
SO4	<1 mg/L	100	90-110
Run No 430996 Analysis/Extraction Date 2022-06-10 Analyst ACG Method SM2320,2510,4500H/F			
Conductivity	<5 uS/cm	101	90-110
pH		100	90-110
Run No 431191 Analysis/Extraction Date 2022-10-12 Analyst AET Method Resistivity - water			
Resistivity			

Guideline = * = **Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX D

Site Photographs



Photograph 1: Looking northwest from the proposed Vehicle Maintenance Building area across the site towards Highway 417 / OR29 Interchange; July 17, 2022



Photograph 2: Looking southwest across the site towards the location of the proposed Vehicle Maintenance Building; July 17, 2022



Photograph 3: Looking southwest across from the location of the proposed Vehicle Maintenance Building towards OR29, Borehole 23-03 and the proposed location of the SWMP; August 17, 2023



Photograph 4: Looking southwest along Upper Dwyer Hill Road; July 17, 2022

APPENDIX E

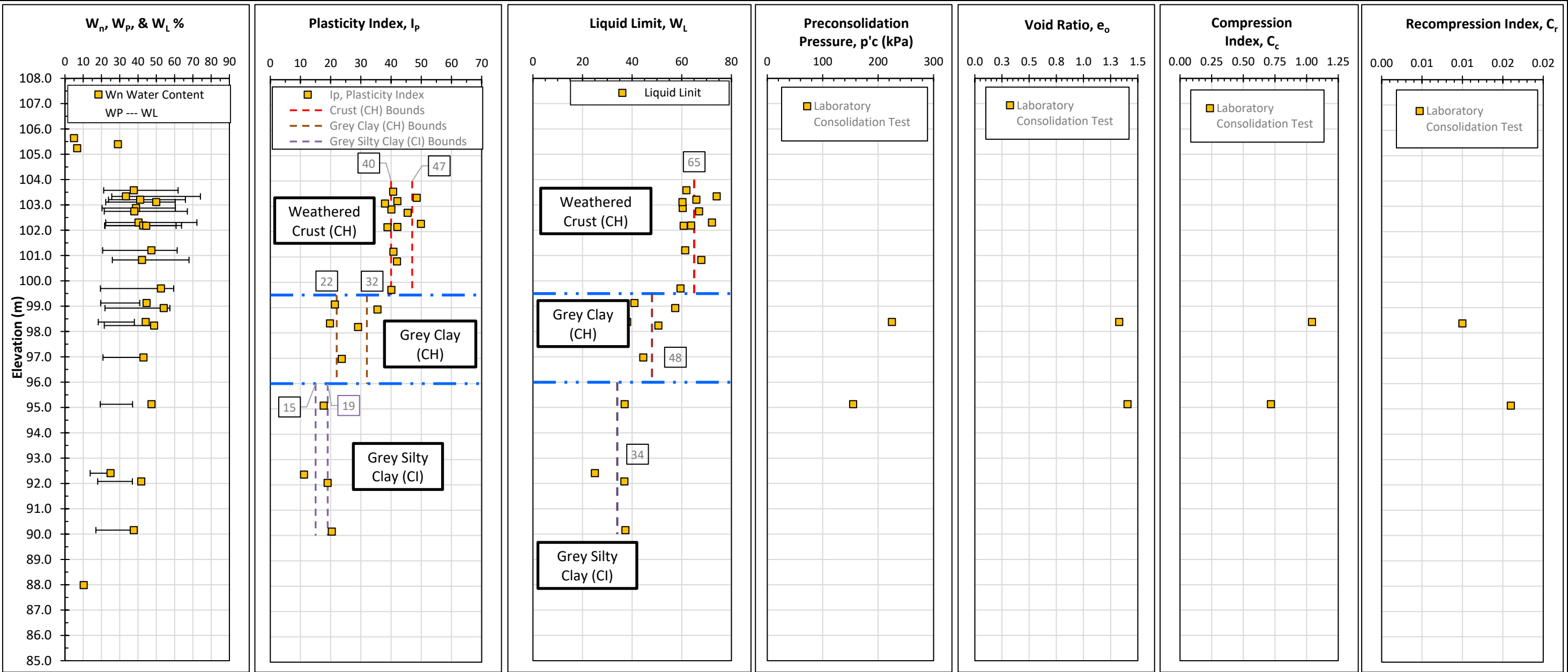
Results of Global Stability Assessment

Figure E1 Corrected Shear Strength

Figure E2 – Undrained Static Slope Stability Assessment

Figure E3 – Undrained Seismic Slope Stability Assessment

Figure E4 – Drained Slope Stability Assessment

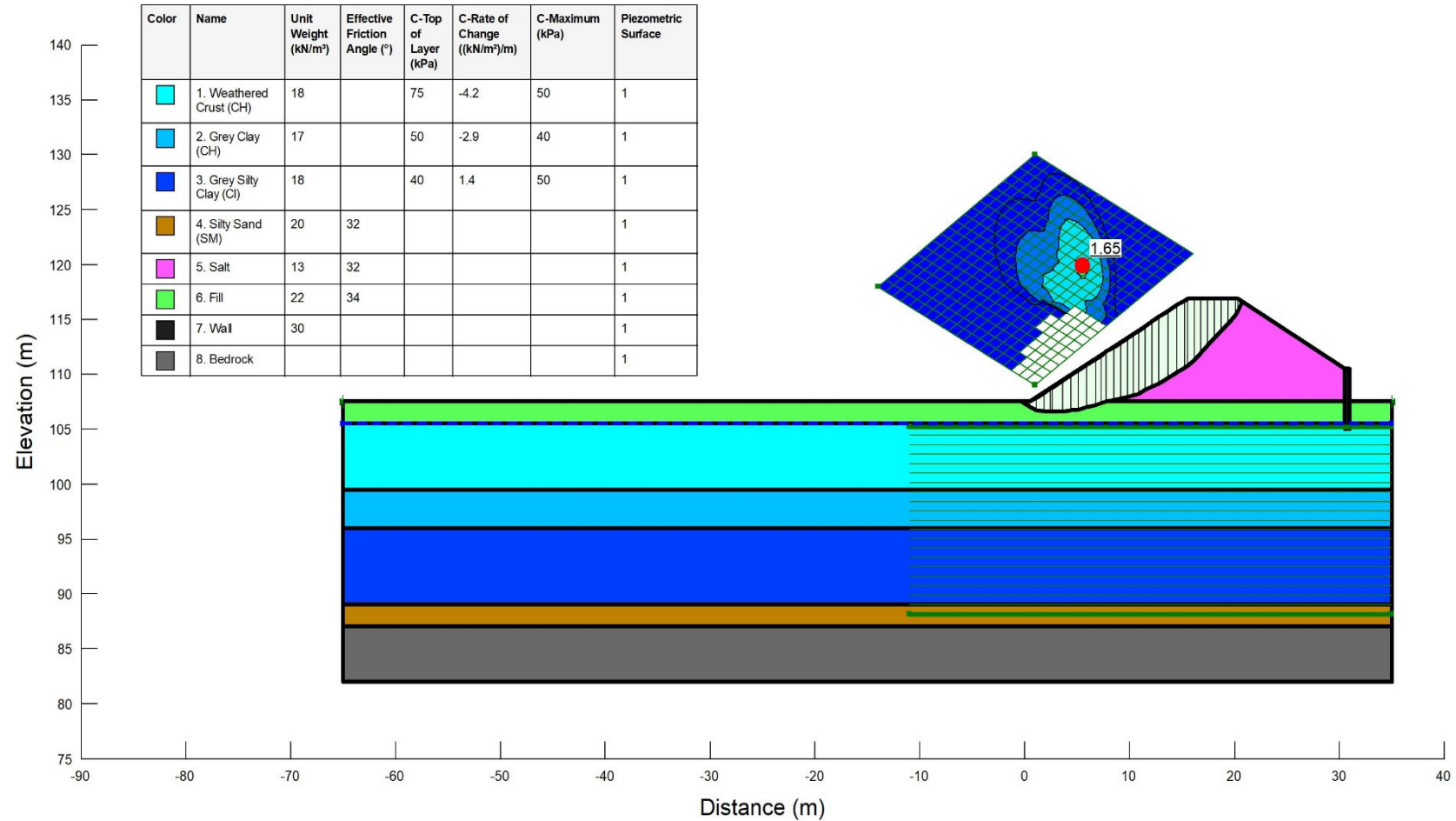


Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Engineering Properties from Laboratory Testing

Project No.: 21480555 - 12298.4565
Date: January 2, 2024
Drawn: KCP
Review: JPD

Figure E1

Title: Arnprior MPY
File Name: Slope W Analysis Jan 04, 2024.gsz
Name: 1-undrained static
Analysis Type: Morgenstern-Price
Direction of movement: Right to Left
Groundwater Elevation: 105.5 m
Horz Seismic Coef.:

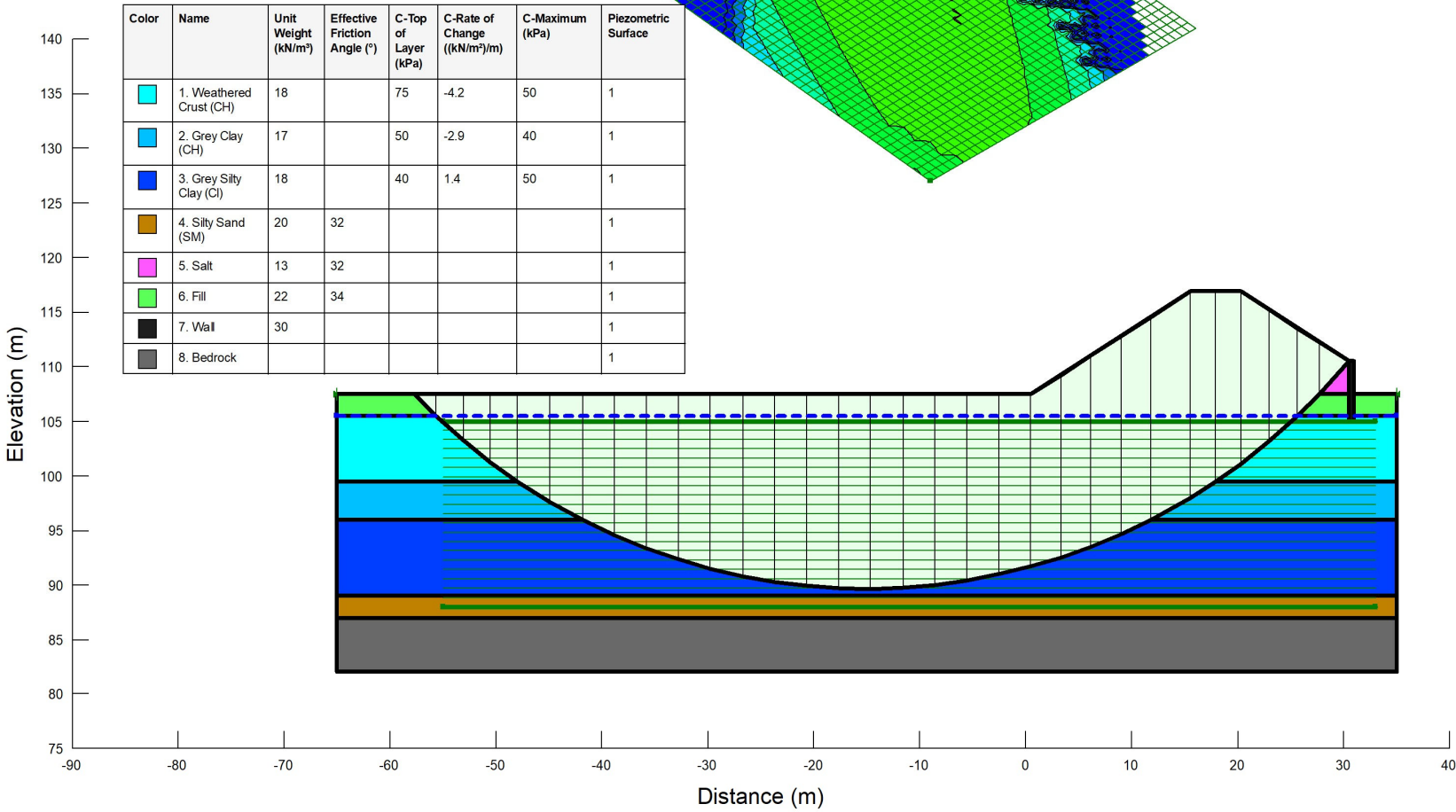
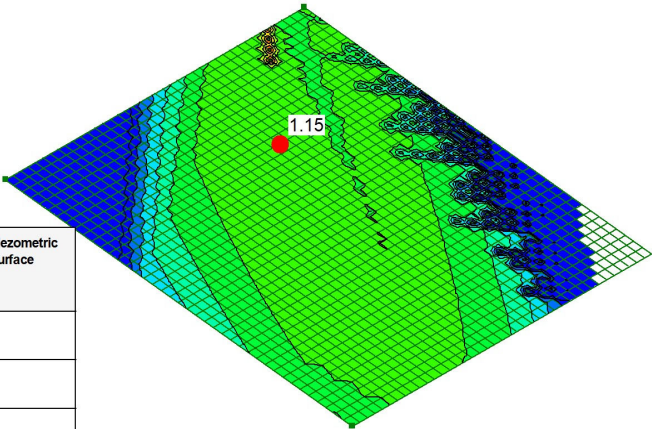


Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Undrained Static Slope Stability Assessment

Project No: 21480555/ CA0012298.4565
Drawn: KCP/KG
Date: January 5, 2024
Checked: KCP
Review: JPD

FIGURE E2

Title: Arnprior MPY
File Name: Slope W Analysis Jan 04, 2024.gsz
Name: 2-Undrained Seismic
Analysis Type: Morgenstern-Price
Direction of movement: Right to Left
Groundwater Elevation: 105.5 m
Horz Seismic Coef.: 0.164

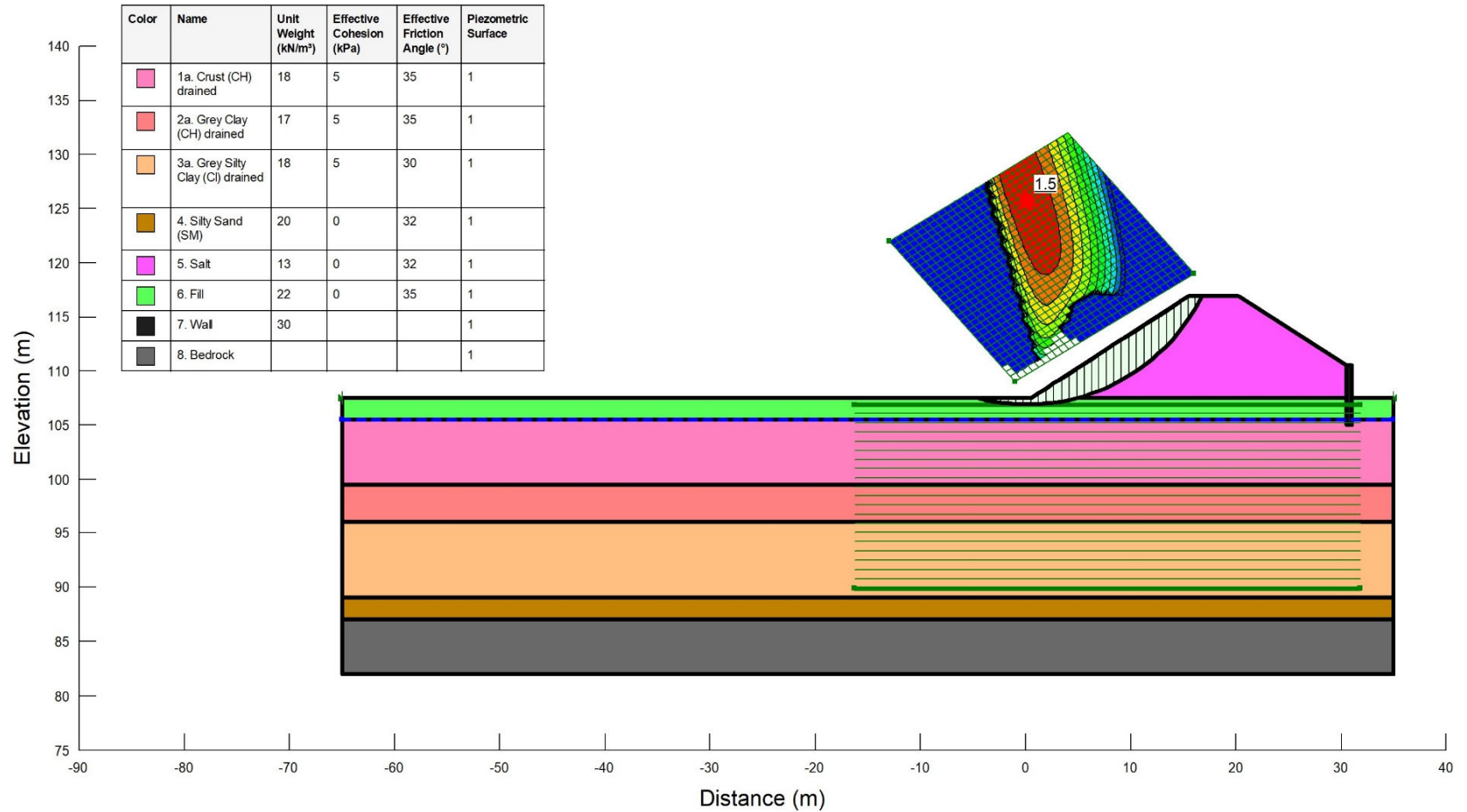


Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Undrained Seismic Slope Stability Assessment

Project No: 21480555/ CA0012298.4565
Drawn: KCP/KG
Date: January 5, 2024
Checked: KCP
Review: JPD

FIGURE E3

Title: Arnprior MPY
File Name: Slope W Analysis Jan 04, 2024.gsz
Name: 3-Drained static
Analysis Type: Morgenstern-Price
Direction of movement: Right to Left
Groundwater Elevation: 105.5 m
Horz Seismic Coef.:



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Drained Slope Stability Assessment

Project No: 21480555/ CA0012298.4565
Drawn: KCP/KG
Date: January 5, 2024
Checked: KCP
Review: JPD

FIGURE E4

APPENDIX F

Results of Settlement Analysis

Figure F1 – Settlement Design Parameters

Figure F2 – Distance Vs Settlement Plot for Query Line 1 along Column Line A-A.1

Figure F3 – Distance Vs Settlement Plot for Query Line 2 along Column Line 2

Figure F4 – Distance Vs Settlement Plot for Query Line 3 along Middle of Salt Pile (West-East)

Figure F5 – Distance Vs Settlement Plot for Query Line 4 along Middle of Salt Pile (North-South)

Figure F6 – Distance Vs Settlement Plot for Query Line 5 along Middle of Sand Pile (North-South)

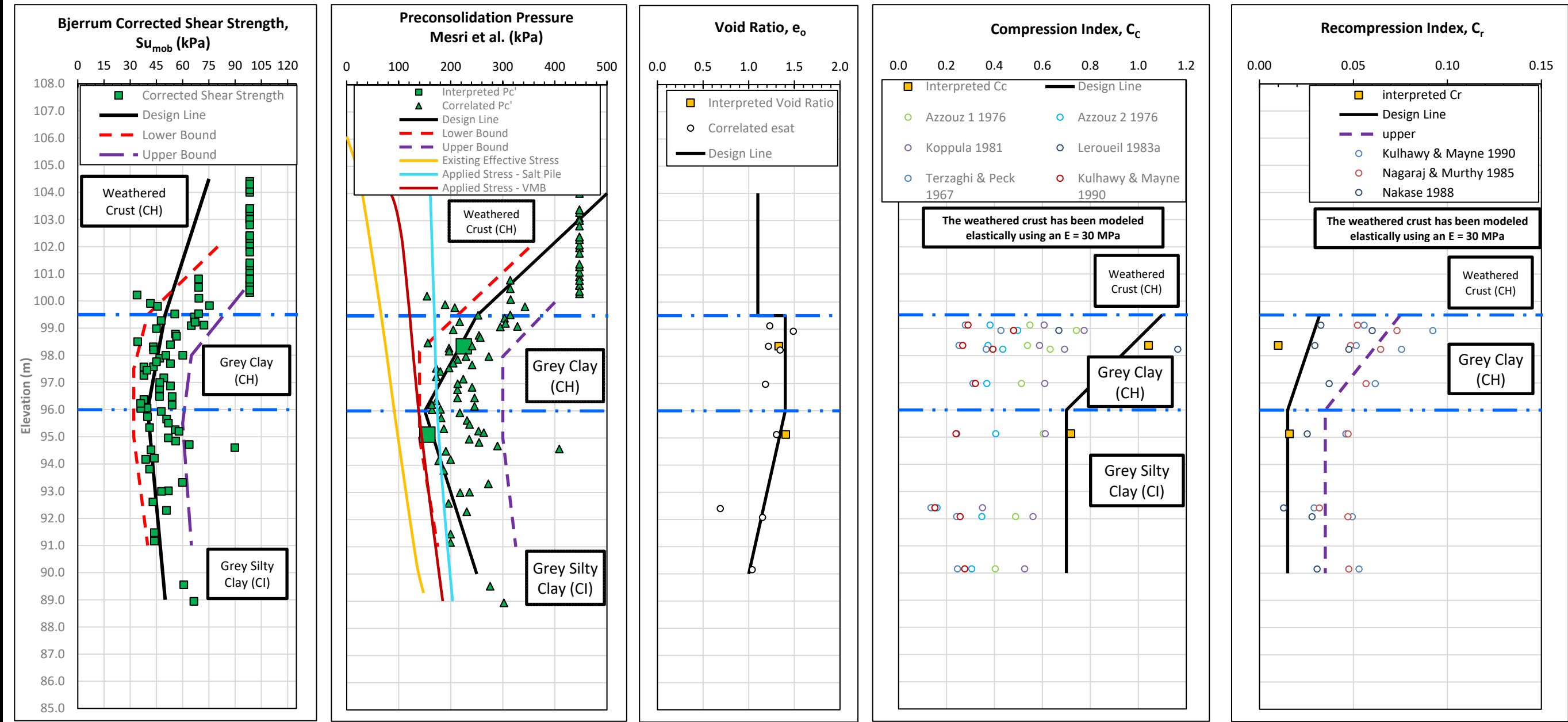
Figure F7 – Distance Vs Settlement Plot for Query Line 1 along Column Line A-A.1

Figure F8 – Distance Vs Settlement Plot for Query Line 2 along Column Line 2

Figure F9 – Distance Vs Settlement Plot for Query Line 3 along Middle of Salt Pile (West-East)

Figure F10 – Distance Vs Settlement Plot for Query Line 4 along Middle of Salt Pile (North-South)

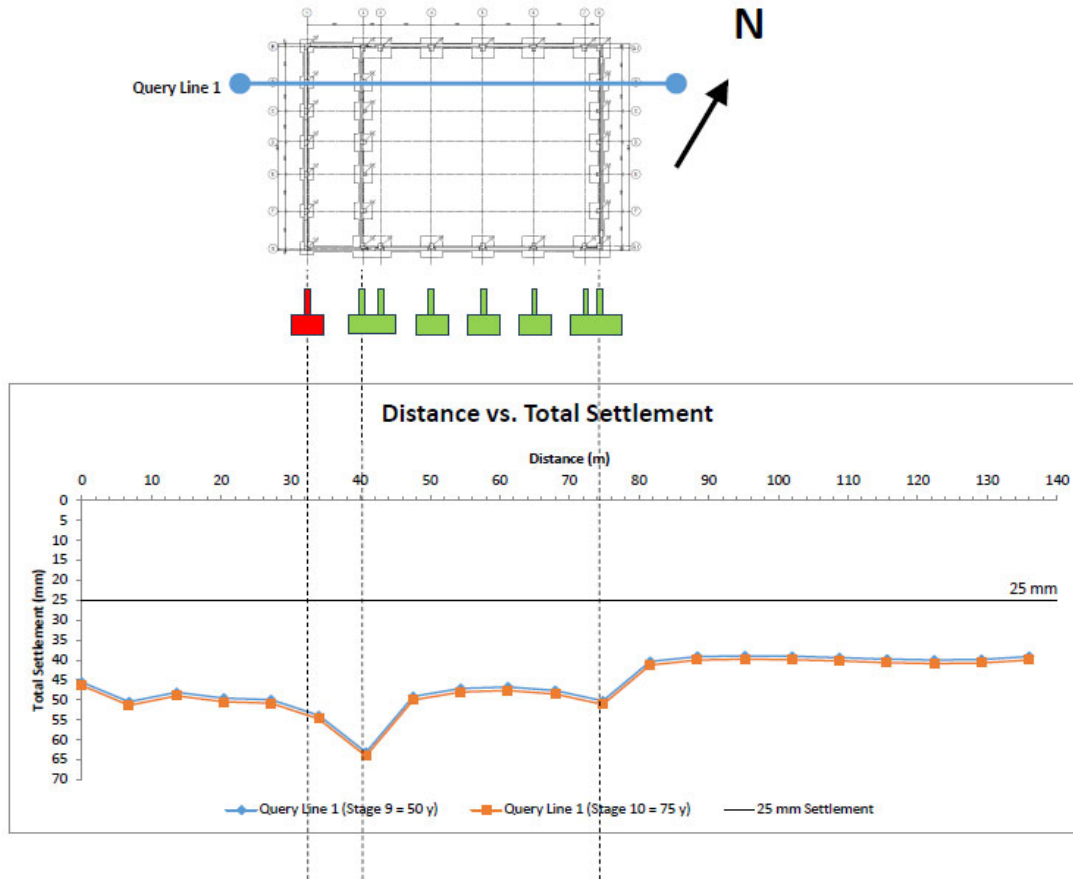
Figure F11 – Distance Vs Settlement Plot for Query Line 5 along Middle of Sand Pile (North-South)



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29, Arnprior, Ontario
Settlement Design Parameters

Project No.: 21480555 - 12298.4565
Date: January 2, 2024
Drawn: KCP
Review: JPD

FIGURE F1



Note:
 Footing shown in Red is Type F1 having size 1.8 m x 1.8 m.
 Footing shown in Green is Type F2 having size 3.0 m x 3.0 m.
 In the analysis, 150 kPa stress is applied to all footings.

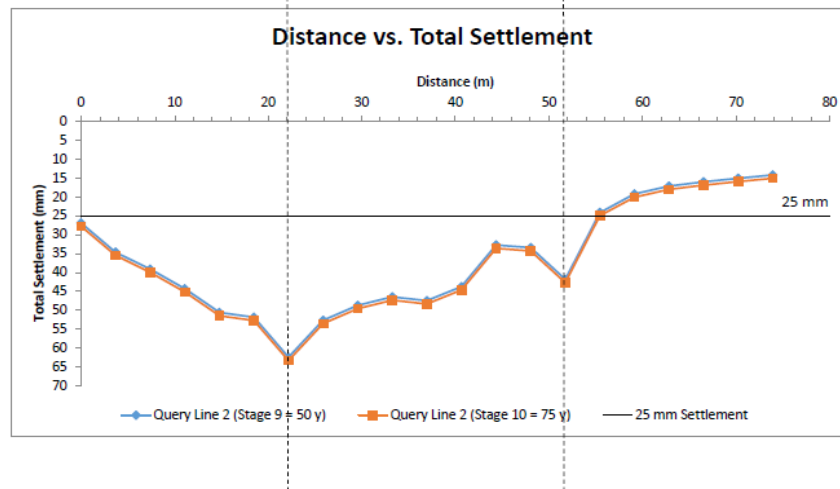
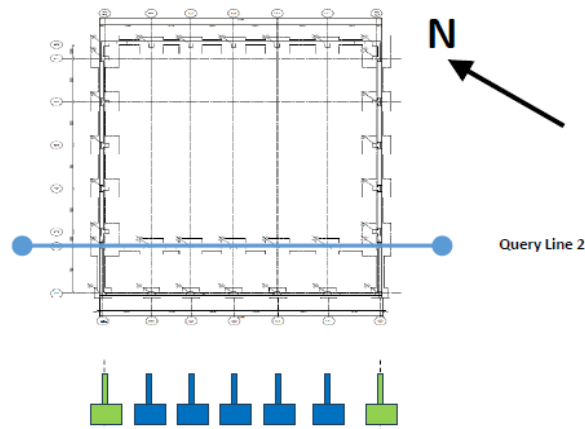
Figure F1: Distance Vs Settlement Plot for Query Line 1 along Column Line A-A.1
Grade Raise and Design Footing Stress of 150 kPa at SLS are Applied together at the Initial Stage of Analysis



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29
Arnprior, Ontario

Project No.: 21480555/ CA0012298.4565
 Drawn: KG
 Date: 2024-01-05
 Review: KCP/JPD
 Date: 2024-01-12

Figure F2



Note:
 Footing shown in Green is Type F2 having size 3.0 m x 3.0 m.
 Footing shown in Blue is Type F3 having size 2.5 m x 2.5 m.
 In the analysis, 150 kPa stress is applied to all footings.

Figure F2: Distance Vs Settlement Plot for Query Line 2 along Column Line 2
Grade Raise and Design Footing Stress of 150 kPa at SLS are Applied together at the Initial Stage of Analysis



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29
Arnprior, Ontario

Project No.: 21480555/ CA0012298.4565
 Drawn: KG
 Date: 2024-01-05
 Review: KCP/JPD
 Date: 2024-01-12

Figure F3

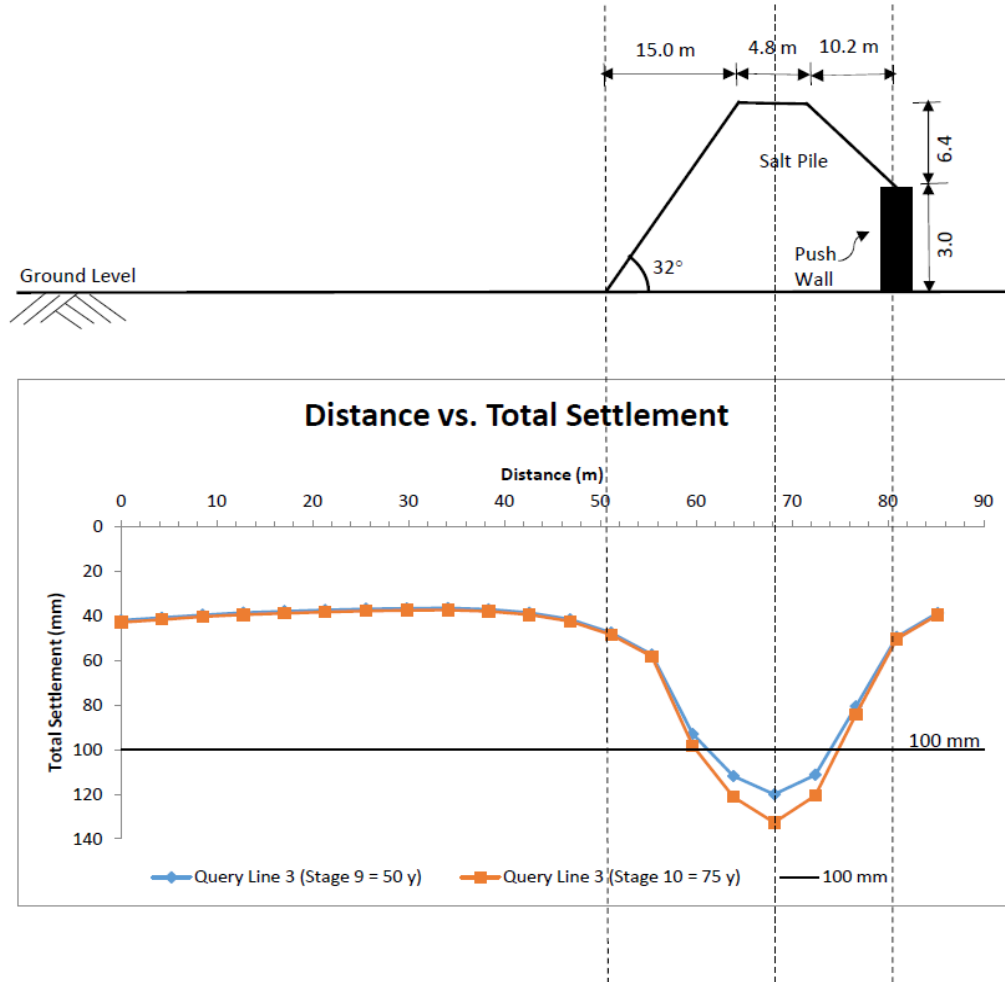


Figure F3: Distance Vs Settlement Plot for Query Line 3 along Middle of Salt Pile (West-East)
Grade Raise and Full Height Salt Pile are Applied together at the Initial Stage of Analysis



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29
Arnprior, Ontario

Project No.: 21480555/ CA0012298.4565
Drawn: KG
Date: 2024-01-05
Review: KCP/JPD
Date: 2024-01-12

Figure F4

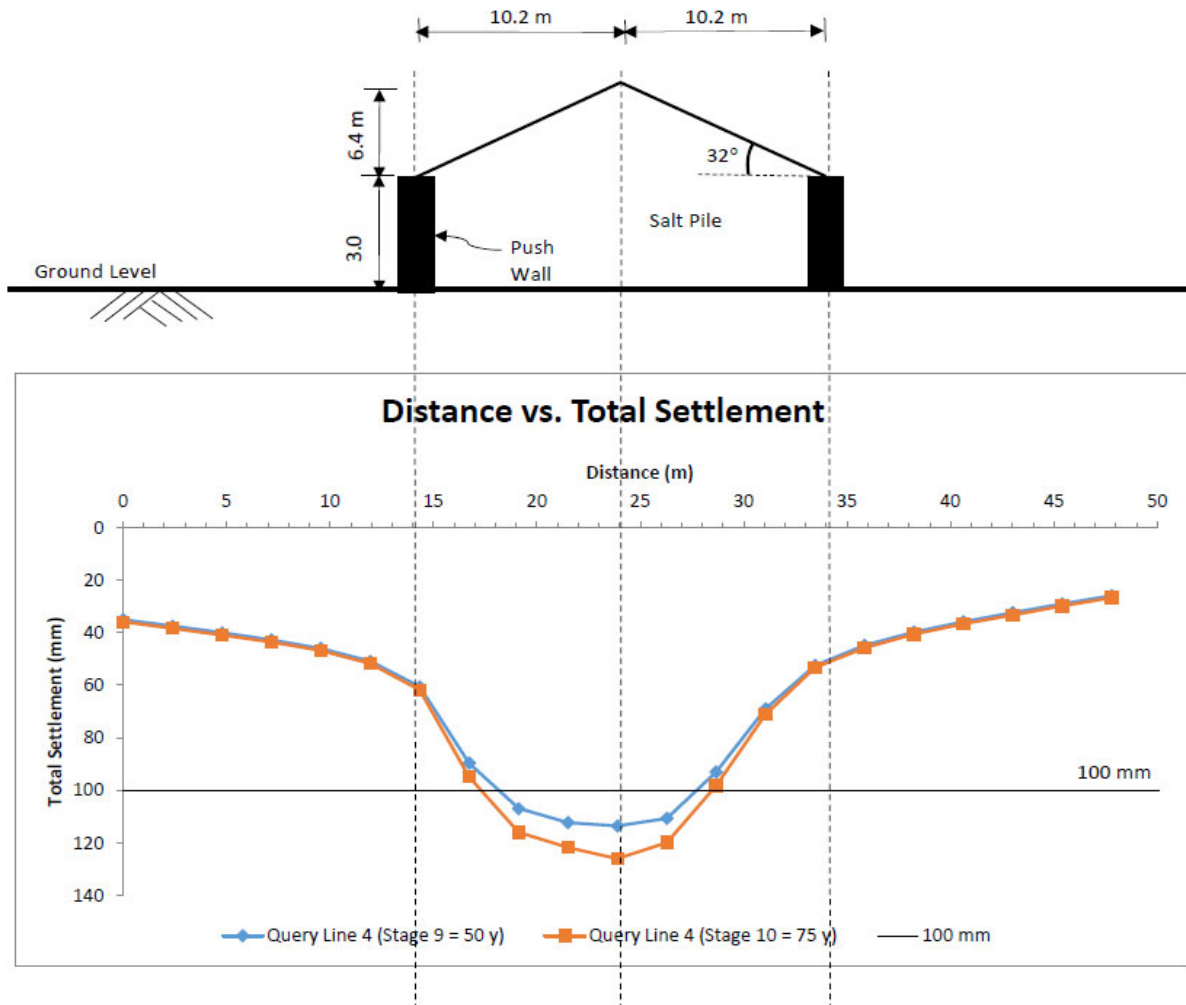


Figure F4: Distance Vs Settlement Plot for Query Line 4 along Middle of Salt Pile (North-South)
Grade Raise and Full Height Salt Pile are Applied together at the Initial Stage of Analysis



Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29
Arnprior, Ontario

Project No.: 21480555/ CA0012298.4565
 Drawn: KG
 Date: 2024-01-05
 Review: KCP/JPD
 Date: 2024-01-12

Figure F5

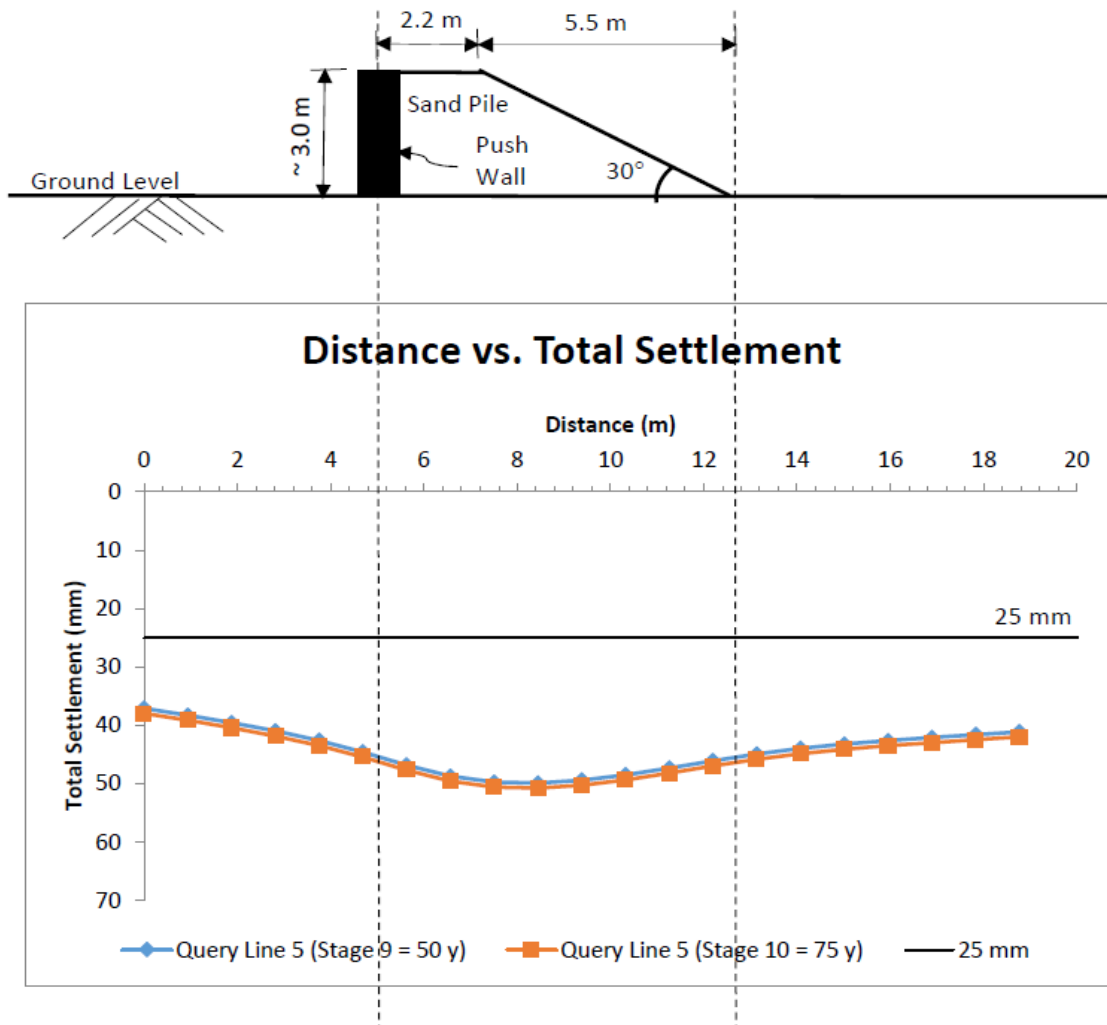


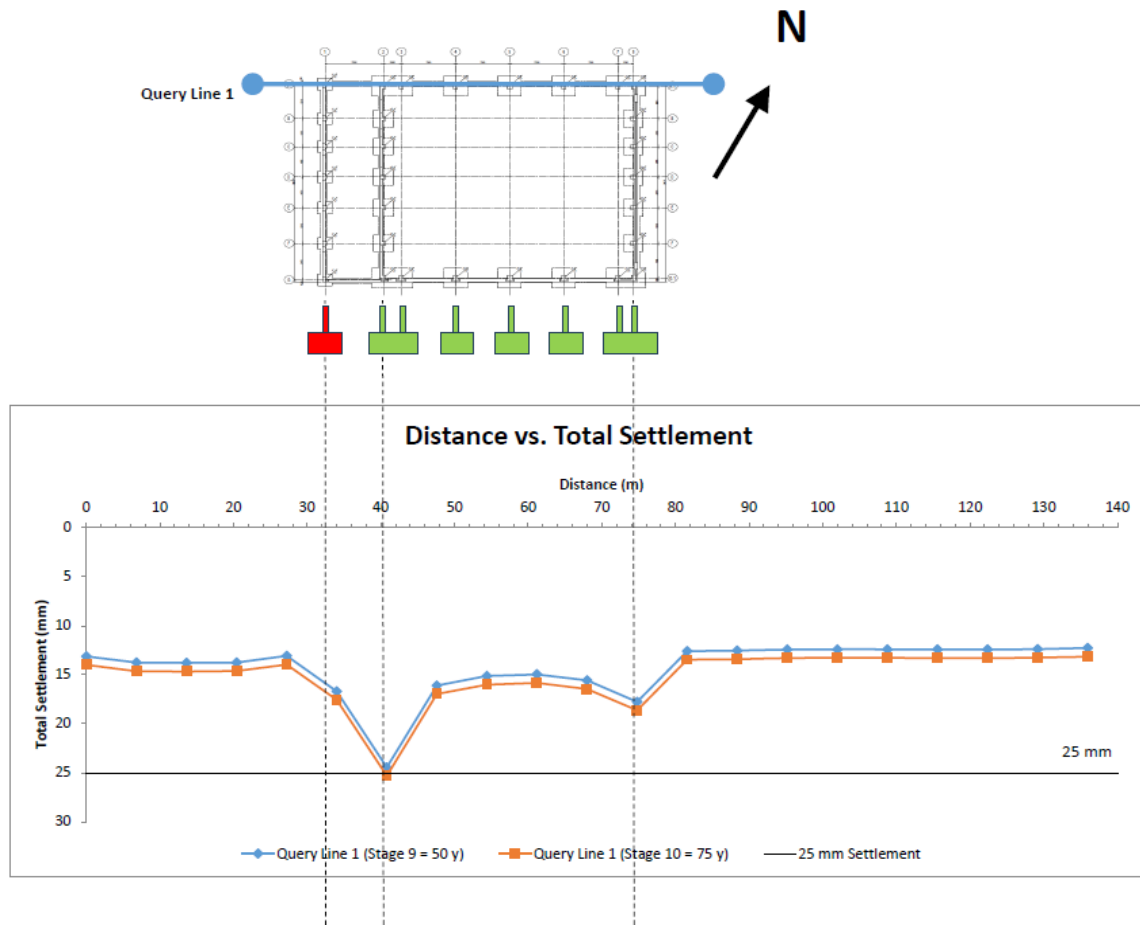
Figure F5: Distance Vs Settlement Plot for Query Line 5 along Middle of Sand Pile (North-South)
Grade Raise and Full Height Sand Pile are Applied together at the Initial Stage of Analysis



Foundation Investigation and Design
 Arnprior Maintenance Patrol Yard
 Location 12- Highway 417 and Ottawa Road 29
 Arnprior, Ontario

Project No.: 21480555/ CA0012298.4565
 Drawn: KG
 Date: 2024-01-05
 Review: KCP/JPD
 Date: 2024-01-12

Figure F6



Note:

Footing shown in Red is Type F1 having size 1.8 m x 1.8 m.

Footing shown in Green is Type F2 having size 3.0 m x 3.0 m.

In the analysis, 100 kPa stress is applied to all footings.

In the analysis, 1 m high surcharge for 7 months is applied around the footprint of VMG.

The above plot illustrates the estimated settlement after the 7 months of surcharge period.

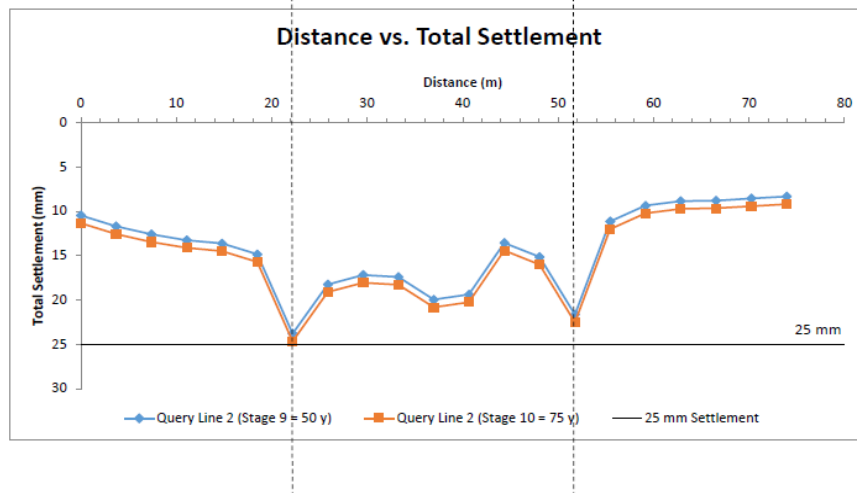
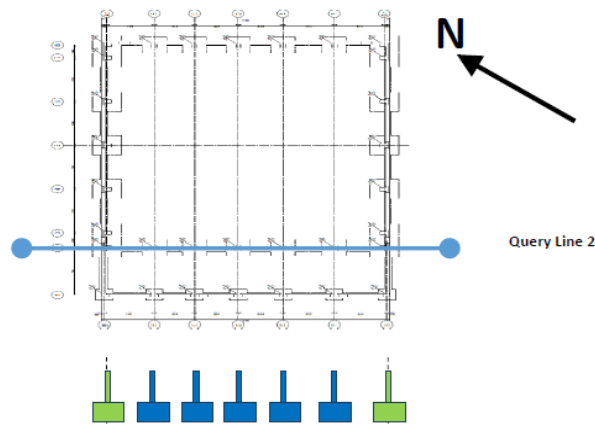
**Figure F6: Distance Vs Settlement Plot for Query Line 1 along Column Line A-A.1
Grade Raise and 1 m High Surcharge Applied for 7 Months followed by Footing Stress of 100 kPa at SLS**



**Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
Location 12- Highway 417 and Ottawa Road 29
Arnprior, Ontario**

Project No.: 21480555/ CA0012298.4565
Drawn: KG
Date: 2024-01-05
Review: KCP/JPD
Date: 2024-01-12

Figure F7



Note:
 The corner footings shown in Green are 2-Type F2 having size 3.0 m x 3.0 m.
 Footing shown in Blue is Type F3 having size 2.5 m x 2.5 m.
 In the analysis, 100 kPa stress is applied to all footings.
 In the analysis, 1 m high surcharge for 7 months is applied around the footprint of VMG.
 The above plot illustrates the estimated settlement after the 7 months of surcharge period.

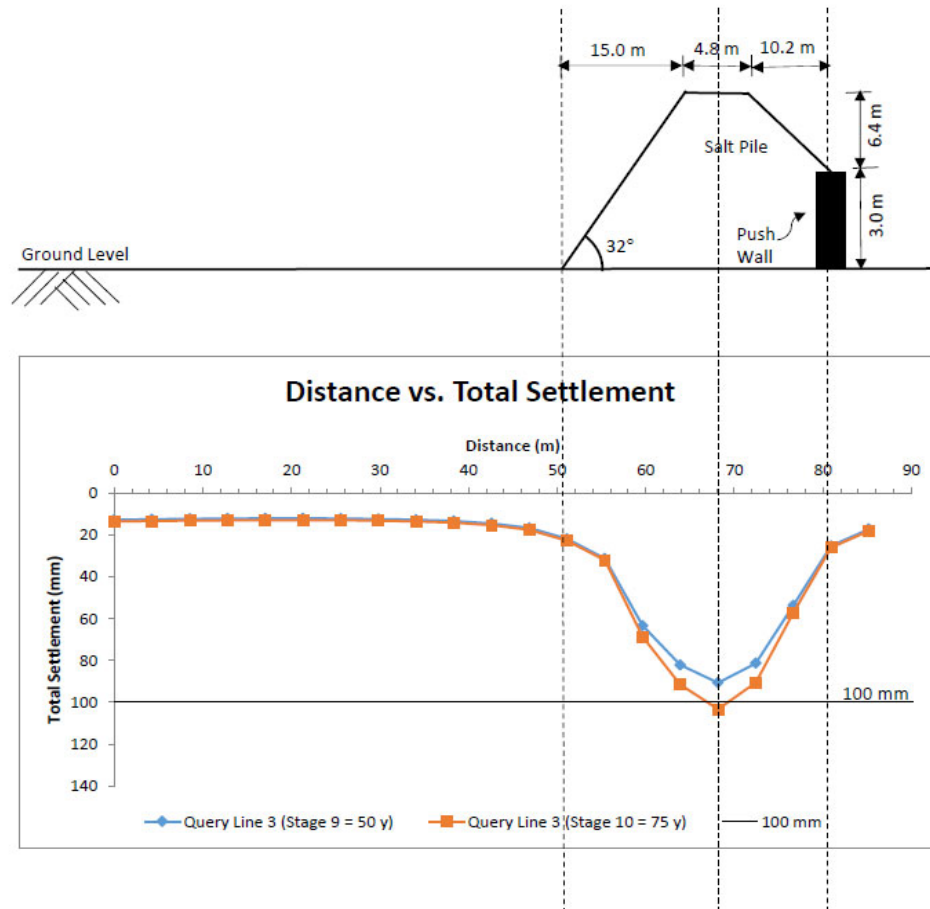
**Figure F7: Distance Vs Settlement Plot for Query Line 2 along Column Line 2
 Grade Raise and 1 m High Surcharge Applied for 7 Months followed by Footing Stress of 100 kPa at SLS**



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Arnprior Maintenance Patrol Yard
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Arnprior, Ontario

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 Drawn: KG
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 Date: 2024-01-12

Figure F8



Note:
The above plot illustrates the estimated settlement after 7 months.

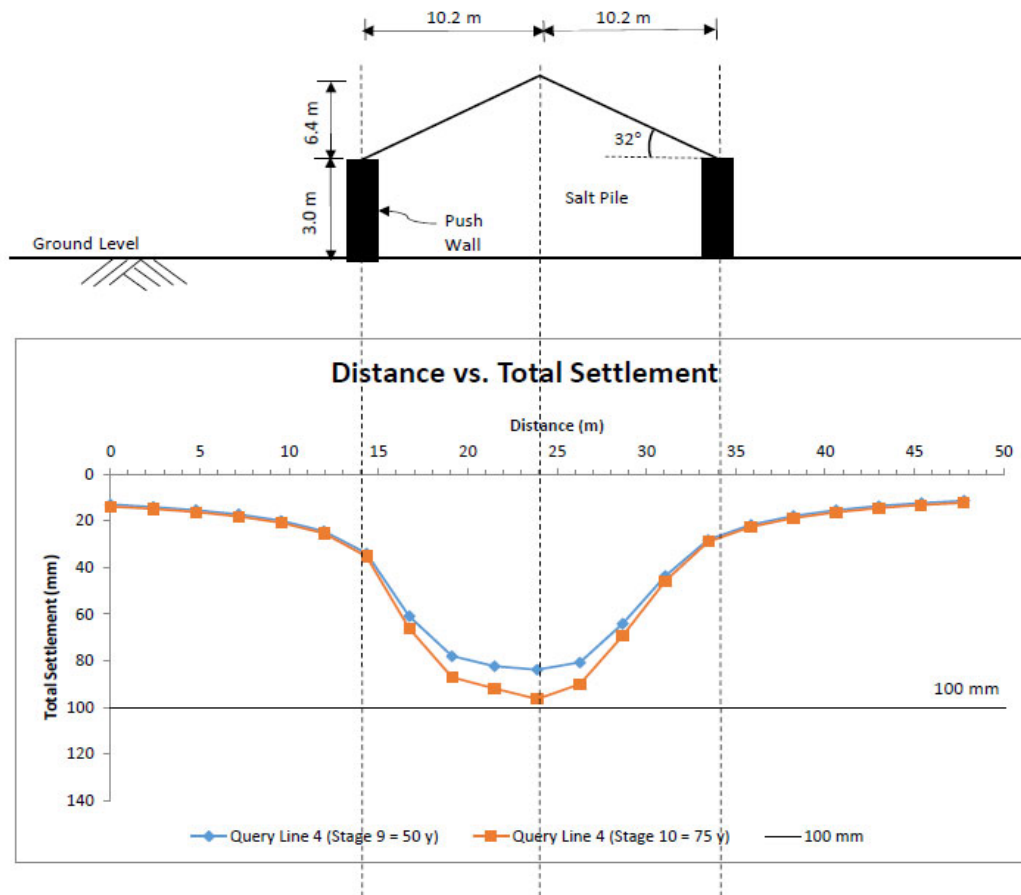
**Figure F8: Distance Vs Settlement Plot for Query Line 3 along Middle of Salt Pile (West-East)
Grade Raise Applied for 7 Months followed by Full Height Salt Pile**



**Foundation Investigation and Design
Arnprior Maintenance Patrol Yard
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Arnprior, Ontario**

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Date: 2024-01-05
Review: KCP/JPD
Date: 2024-01-12

Figure F9



Note:
The above plot illustrates the estimated settlement after 7 months.

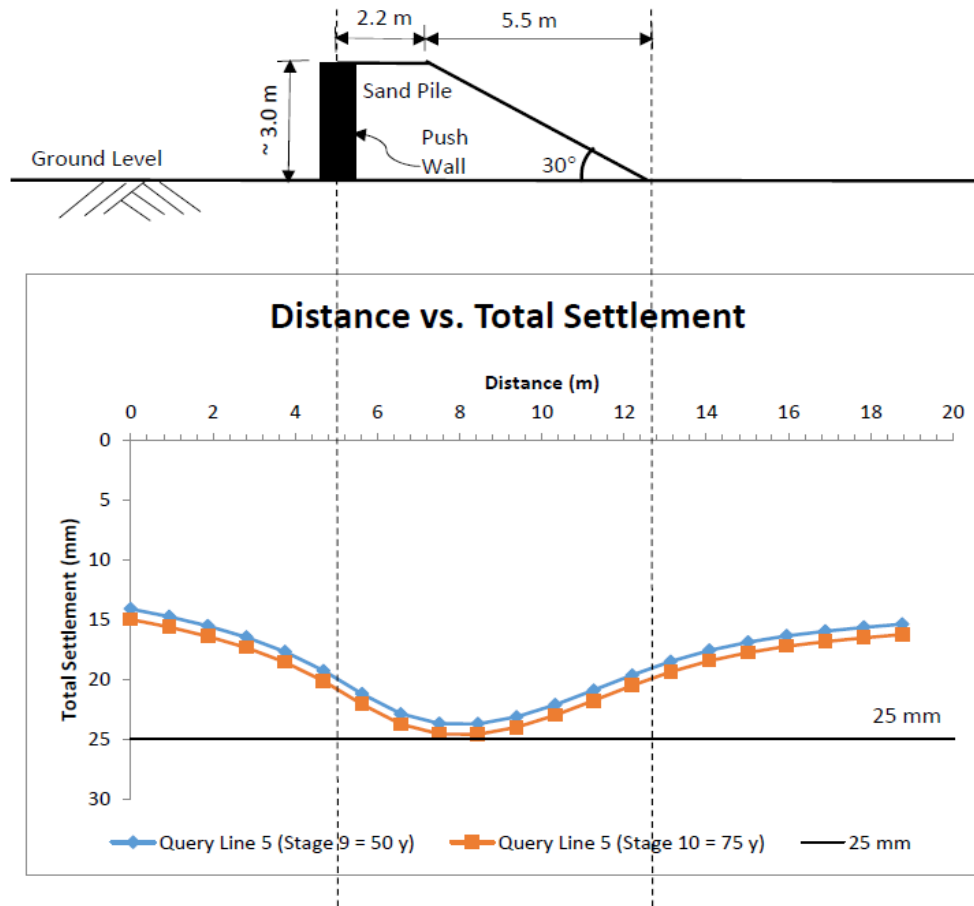
**Figure F9: Distance Vs Settlement Plot for Query Line 4 along Middle of Salt Pile (North-South)
Grade Raise Applied for 7 Months followed by Full Height Salt Pile**



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Arnprior Maintenance Patrol Yard
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Arnprior, Ontario

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Figure F10



Note:
The above plot illustrates the estimated settlement after 7 months.

**Figure F10: Distance Vs Settlement Plot for Query Line 5 along Middle of Sand Pile (North-South)
Grade Raise Applied for 7 Months followed by Full Height Sand Pile**



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 Drawn: KG
 Date: 2024-01-05
 Review: KCP/JPD
 Date: 2024-01-12

Figure F11

APPENDIX G

Non-Standard Special Provisions

Concrete Working Slab

Protection of Sensitive Foundation Soils

Cobble and Boulder Obstructions and Sloping Bedrock

Supply and Installation of Grade Raise and Surcharge Monitoring Equipment

Drawing 4 – Settlement Monitoring Plan

Drawing 5 – Typical Embankment Monitoring Instrumentation Details

Contract Administration Terms of Reference

Foundation Engineering Specialist for Foundation Monitoring Program – Contract 2024–4033

NON-STANDARD SPECIAL PROVISIONS

RECOMMENDED WORDING FOR “NSSP – A CONCRETE WORKING SLAB”

This Non-standard Special Provision covers the requirements for the supply and placement of a concrete working slab to protect the clay subgrade to provide a proper working surface for the installation of the foundations.

Excavation for the working slab shall be according to OPSS.PROV 902. Within four hours following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents. Concrete for working slabs shall have a minimum 28-day strength of 20 MPa.

RECOMMENDED WORDING FOR “NSSP – Protection of Sensitive Foundation Soils”

The Contractor is advised that the soil that will be exposed at the subgrade during the construction of the foundations is moisture sensitive and may become disturbed or otherwise negatively impacted when subjected to construction or personal traffic, freeze thaw actions, ingress or ponding of water. The Contractor shall be responsible for implementing adequate groundwater control measures and to minimize construction and personnel traffic on the founding subgrade.

RECOMMENDED WORDING FOR “NSSP Cobble and Boulder Obstructions and Sloping Bedrock”

The contractor shall be alerted that the overburden soils at the site are glacially derived materials and contain cobbles and boulders. The contractor shall also be alerted that the bedrock surface at the proposed material storage building is variable and sloping. Any foundations designed on bedrock should account for the varying founding elevations, pile lengths, etc. Consideration must be given to potential difficulties driving through the till containing cobbles and boulders as well as the potential difficulties seating the piles during driving due to the presence sloping bedrock at this site. As a result, piles should be fitted with appropriate driving shoes and/or rock point as per OPSS.PROV 903 Section 903.07.02.02 Driving Shoes and Rock Points; the APF Hard-Bit points (Model: HP-77750-B) or equivalent are recommended for this site.

SUPPLY AND INSTALLATION OF GRADE RAISE AND SURCHARGE MONITORING EQUIPMENT – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

This special provision describes requirements to supply and install monitoring instrumentation to verify the response of the foundation soils to the construction of the new Arnprior Maintenance Patrol Yard (MPY), specifically the vehicle maintenance and administration building at Highway 417 and Ottawa Road 29 in Arnprior Ontario. This includes monitoring the effects of the soil grade raise and surcharge construction to settlements and pore pressures in the foundation soils using the following geotechnical monitoring instrumentation:

- Settlement Plates (SP);
- Vibrating Wire Piezometers (VWP).

This special provision also contains the requirements for the supply and installation of Temporary Survey Benchmarks (TBMs) related to the geotechnical monitoring instrumentation described herein.

1.2 Purpose

The purpose of these instruments and equipment is to monitor the progress of settlement and pore pressure development and dissipation in the foundation soils under and adjacent to the proposed vehicle maintenance and administration building (VMB) at the Arnprior Maintenance Patrol Yard (MPY). The purpose of the temporary survey benchmarks is to provide non-settling reference points for the surveying of the monitoring instruments.

The duration of the surcharging period prior to constructing the VMB will be controlled by the instrumentation readings, as specified elsewhere in the Contract Documents. The completed surcharge within the footprint of the VMB shall remain undisturbed until such time as the monitoring indicates that a sufficient degree of consolidation (i.e., minimum 70%) of the foundation soils has been achieved. The surcharge period is estimated to be approximately seven months long following completion of the surcharge construction to achieve the target degree of consolidation.

1.3 Personnel

The Contractor shall carry out the supply and installation of the geotechnical monitoring instrumentation (temporary benchmarks, settlement plates, and vibrating wire piezometers) in accordance with this Special Provision and the Contract Drawings. The installation of the instruments and set-up for monitoring shall be carried out by a Foundation Engineering consultant (Contractor's Engineer) retained by the Contractor and registered in MTO's Consultant Registry, Appraisal and Qualifications System (RAQS) for "Geotechnical (Structures and Embankments) – Medium Complexity" or higher. All surveying shall be carried out by a qualified and registered surveyor retained by the Contractor.

2.0 REFERENCES

2.1 General

When the Contract Documents indicate that provincial oriented specifications are to be used and there is a provincial oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be the OPSS listed, unless use of a municipal oriented specification is specified in the Contract Documents.

This Special Provision refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction

OPSS 905 Steel Reinforcement for Concrete

Ontario Provincial Standards Specifications, Material

OPSS.PROV 1010 Aggregates Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1350 Concrete – Materials and Production

OPSS.PROV 1205 Clay Seal

OPSS.MUNI 1301 Cementing Materials

OPSS.PROV 1302, Water

OPSS.PROV 1801 Corrugated Steel Pile (CSP) Products

Ontario Water Resources Act RRO 1990:

Regulation 903 Wells (as amended)

2.2 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

Foundation Investigation and Design Report Detailed Design Proposed Arnprior Maintenance Patrol Yard
Location 12 - Highway 417 and Ottawa Road 29, Arnprior, Ontario MTO Agreement No. 4020-E-0012-32;
GEOCREs No.: 31F08-001

3.0 DEFINITIONS

Contractor's Engineer (CE): An engineering firm that is registered with MTO RAQS "Geotechnical (Structures and Embankments) – Medium Complexity" or higher. It is expected that the selected firm will have carried out at least four (4) similar instrumentation installation projects for the MTO within the last ten (10) years. The CE shall be retained by the Contractor to coordinate / support the supply and installation of the instrumentation, complete baseline monitoring, and all associated reporting as outlined herein.

Equal shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

Foundation Engineering Specialist (FES): An engineering firm that is registered with MTO RAQS “Geotechnical (Structures and Embankments) – High Complexity”. The FES shall be retained by the Contract Administrator (CA) to ensure general conformance with the contract documents, to carry out the monitoring and interpretation of the instrumentation data and shall issue certificate(s) of conformance.

Monitoring Program means the monitoring readings conducted by others as part of the Contract Administration Assignment.

Settlement Plate means a plate installed at the defined level with a series of rods attached to the plate for the purposes of settlement monitoring.

Temporary Survey Benchmark means a non-yielding, deep-seated survey reference point.

Vibrating Wire Piezometer means a sensor attached to a cable installed in, or below, a borehole for the purposes of measuring pore water pressure response.

4.0 SUBMISSION AND DESIGN REQUIREMENTS

4.1 Submission Requirements

4.1.1 Notification

The Contract Administrator shall be notified a minimum of fifteen (15) working days in advance of commencing the installation of instruments.

4.1.2 Installation Methods

The Contractor shall submit details of the proposed installation methods including locations and types of the data logger(s), monitoring equipment protection/enclosure(s), temporary survey benchmarks and installation schedule, to the Contract Administrator, a minimum of fifteen (15) working days before the start of instrument installation.

5.0 MATERIALS

5.1 Materials for Temporary Benchmarks (TBM)

The Contractor shall supply all materials and equipment required for the installation of the TBMs as listed below.

5.1.1 Rod

The Contractor shall supply a steel pipe, Schedule 40, with an outside diameter not less than 25.4 mm (1”), supplied in lengths as required to complete the installation as described in Section 7.2.2.

The top end of each length of TBM rod shall be threaded to receive a cap or to allow for connection of successive lengths of rods. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to for successive readings.

5.1.2 Cap

A rounded cap shall be installed at the top of the upper rod for the long-term monitoring to be carried out following completion of construction in such a way that a single survey point can be clearly identified and returned to for successive readings.

5.1.3 Rod Anchor Grout

The Contractor shall supply cement-bentonite grout to fix/anchor the bottom of the TBM rod in the borehole. A suitable grout mix design shall consist of 23 kg of bentonite (OPSS.PROV 1205), 49 litres of water (OPSS.PROV 1302), and 40 kg of cement (Type GU – OPSS.MUNI 1301) having a minimum compressive strength of 10 MPa and prepared in accordance with OPSS.MUNI 1350.

5.1.4 Friction-Reducing Sleeve

The Contractor shall supply a friction-reducing sleeve consisting of Schedule 40 – 50.8 mm (2") outer diameter PVC pipe cut perpendicular to the axis of the pipe.

5.1.5 Sand

The annular space between the borehole wall and the friction-reducing sleeve shall be filled with sand. The Contractor shall supply clean, washed sand. The sand shall be Sakcrete washed general purpose sand – or equal.

5.1.6 Flush-Mount, Water-Tight Casing

The Contractor shall supply a lockable, flush-mount, water-tight casing for security/protection of the TBM for the long-term monitoring to be carried out following completion of construction.

5.2 Materials for Settlement Plate (SP)

The Contractor shall supply all materials and equipment required for the installation of the Settlement Plates.

5.2.1 Plate

The Contractor shall supply a steel plate with thickness of at least 6.35 mm at the locations shown in the Contract Drawings. The plate shall be at least 0.5 m by 0.5 m in plan dimensions.

5.2.2 Rod

The Contractor shall supply a steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1"), supplied in lengths as required to complete the installation.

The top end of each length of rod shall be threaded to receive a coupler, or a cap.

5.2.3 Cap

A rounded cap shall be installed at the top of the upper rod for the SPs in such a way that a single survey point can be clearly identified and returned to.

5.2.4 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 - 50.8 mm (2") O.D. PVC pipe cut perpendicular to the axis of the pipe.

5.2.5 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the new fill and extending above the top of the highest fill grade. The surround shall consist of 300 mm diameter corrugated steel pipe (OPSS.PROV 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges.

5.2.6 Sand

The space between the CSP and the Friction Reducing Sleeve (PVC pipe) shall be filled with sand.

5.2.7 Above-Ground, Lockable Well Cover

The Contractor shall supply an above-ground, lockable well cover for security/protection of the temporary SPs during construction monitoring. The cover shall be lockable, tamper resistant, and weather resistant.

5.3 Materials for Vibrating Wire Piezometer (VWP)

The Contractor shall supply all materials and equipment required for the installation of the Vibrating Wire Piezometer.

The Contractor shall submit a detailed proposal on the setup of the datalogging system (i.e., number and location of the datalogging unit(s)) to the CA for review and approval, prior to ordering the data-logger(s).

5.3.1 Vibrating Wire Piezometer

Vibrating Wire Piezometer (Push-In) – Vibrating wire piezometers shall be Durham Geo Slope Indicator (DGSI) Product No. 52621020 (Push-In, 3.5 bar / 50 psi), or approved equivalent. The VWP is to work within the temperature range of -20°C to 80°C.

All VWPs shall be of the same make/supplier. All VWPs shall be calibrated prior to installation and the calibration data for each piezometer shall be provided to the Contract Administrator.

5.3.2 Signal Cable

The Contractor shall supply DGSI Product No. 50613824 standard signal cable, or approved equivalent. The length of cable for each piezometer shall be carefully estimated from the construction drawings to ensure that there is enough signal cable for each piezometer to provide enough slack in the borehole and from the top of borehole to at least 1.5 m above the top of highest fill grade.

5.3.3 Grout

The Contractor shall supply cement-bentonite grout for backfilling. A suitable grout mix design consists of 23 kg of bentonite (OPSS.PROV 1205), 143 litres of water (OPSS.PROV 1302) and 40 kg of cement (Type GU – OPSS.PROV 1301).

5.3.4 Protective Conduit

The signal cable for each VWP shall be contained in a protective conduit (with caps) from the subgrade to at least 1 m above the highest fill grade. The Contractor shall supply suitable conduits to contain the signal cables below and above ground surface (i.e., Schedule 40 – 150 mm dia. steel pipe; or Schedule 80 – 150 mm dia. rigid PVC pipe).

5.3.5 Protective Surround

The Contractor shall supply a protective surround for the portion of the cables and conduit within the new fill and above the top of the highest fill grade. The protective surround shall consist of 300 mm diameter corrugated steel pipe (CSP – OPSS.PROV 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the conduit shall be filled with bentonite gravel.

5.3.6 Bentonite Gravel

The space between the VWP cables and the conduit, and between the conduit and the CSP protective surround shall be filled with bentonite gravel. The Contractor shall supply BAROID Holeplug 3/8” bentonite chips, or equivalent.

5.3.7 Above-ground, lockable well casing

The Contractor shall supply an above-ground, lockable well cover for security/protection of the temporary VWPs during construction monitoring. The cover shall be lockable, tamper resistant, and weather resistant.

5.3.8 Datalogger

The standard signal cables from each of the vibrating wire piezometers shall be connected to a single channel datalogger. The Contractor shall supply DGSI Vibrating Wire Miniloggers (Product No. 52613310), or approved equivalent. The datalogger shall fit inside the lid of the lockable well cover and shall be programmed to record data twenty-four (24) times a day (i.e., 1 reading every 1 hour).

5.3.9 Software

Logger Manager software, or approved equivalent, shall be provided to the Consultant at the completion of the installation. Once this program is transferred to the datalogger, the system shall be tested to confirm readings can be gathered manually at the site.

6.0 EQUIPMENT

6.1 Monitoring Equipment Operation and Weather Conditions

All monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within the ground or on the surface. The instruments shall be capable of operating within the manufacturer's stated accuracy throughout the temperature range. Monitoring will be conducted potentially year-round by the Contract Administrator.

7.0 CONSTRUCTION

7.1 Monitoring Instrument Installations

7.1.1 Drawings

Reference shall be made to the following drawings that are contained elsewhere in the Contract Documents:

- Monitoring Instrumentation Plan and Section; and,
- Typical Monitoring Instrument Installation Details.

7.1.2 Quantities and Locations of Instruments

The quantities and approximate locations of instruments are presented in Table 1 and Table 2 and are shown on the Contract Drawings. The final locations shall be "field fit" by the Contractor to take account of any utilities that may be present, construction operations, and safe access conditions.

Table 1: Instrument Location and Quantities

Location		Monitoring Type	Name	Quantities	Tip Elevation (m)
Northing (m)	Easting (m)				
5030649.994	318064.093	Settlement Plate	SP24-01	1	N/A
5030668.575	318084.844	Settlement Plate	SP24-02	1	N/A
5030653.398	318098.527	Settlement Plate	SP24-03	1	N/A
5030634.596	318077.799	Settlement Plate	SP24-04	1	N/A
5030659.253	318074.434	Vibrating Wire Piezometer	VW24-01	1	97.0
5030648.986	318083.627	Vibrating Wire Piezometer	VW24-02	1	96.5

7.1.3 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless otherwise noted.

7.1.4 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground elevation at each instrument location.

7.1.5 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor at no cost to the Owner or Contract Administrator.

7.1.6 Marking and Labelling

The location of any above-ground monitoring fixture shall be made clearly visible to nearby construction traffic before, during and after grade raise and surcharge construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls, if and where applicable.

Instruments shall be clearly labelled in the field, with each instrument having a unique identifier. The labelling shall remain legible for the entire duration of monitoring.

7.1.7 Protection of Instruments

The Contractor shall adequately protect all instruments such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced by the Contractor at no cost to the Owner or Contract Administrator.

7.1.8 Survey Personnel

Surveying to establish the benchmark(s) and other elevations shall be carried out by a registered surveyor with appropriate equipment. The surveyor shall be retained by the Contractor.

7.1.9 Accuracy of Surveying for Elevations

Elevations shall be surveyed to an accuracy of ± 2 mm or better.

7.1.10 Boreholes

The Contractor shall make a basic stratigraphic log of boreholes as they are being drilled for the installation of monitoring instruments. In-situ or laboratory geotechnical testing is not required.

Boreholes shall be advanced to the required depths using conventional drilling methods, where applicable, and shall be as straight and vertical as practicable.

7.1.11 Installation Program

The instruments shall be installed prior to the commencement of the grade raise / surcharge construction. Table 2

gives a summary of the installation schedule requirements.

Table 2: Instrument Installation Program

Instrument Type	Instrument Location	Start Installation	Finish Installation
SP	Beneath grade raise and surcharge material within the footprint of the proposed VMB Rods extended to above fill surface during grade raise and surcharge construction.	Following removal of organics and prior to placement of any fill	At completion of surcharge construction.
VWP	Beneath grade raise and surcharge material within the footprint of the proposed VMB Cables and casing extended to above fill surface during grade raise and surcharge construction.		

7.2 Temporary Benchmark Installation

7.2.1 General Procedure

The benchmark(s) shall be installed in a borehole and anchored into the bedrock located approximately at Elevations 88 m to 90 m approximately 14 m to 16 m below the existing ground surface at the VMB.

7.2.2 Number and Locations

The minimum number and approximate locations of the benchmark(s) are to be determined by the Contractor and the Contractor's Engineer in conjunction with the Contract Administrator, the Foundation Engineering Specialist (FES), and Surveyors. For bidding purposes assume that one (1) benchmark is required: all benchmarks are to be anchored into the bedrock at Elevations 88 m to 90 m. The number and locations of benchmarks shall be determined in the field to satisfy the following conditions:

- Direct sighting is possible from all instruments to at least one benchmark.
- Each benchmark is to be located so that it will not experience a change in loading (due to grade raise or excavation) that could induce settlement or heave in the ground in which the benchmark is installed (i.e., non-settling benchmark).
- Each benchmark is to be located to minimize interference with and/or damage by construction activities. The rod anchor elevation shall be adjusted in the field to extend to bedrock. Reference shall be made to the Foundation Investigation Reports for information to determine the anchor elevation for each benchmark location selected.

Intermediate tie-in points may be required as deemed necessary by the surveyor and shall be tied into the

temporary benchmarks during each reading.

7.2.3 Installation

The Contractor shall install benchmarks as shown on the Contract Drawings and the Typical Monitoring Instrument Installation Details, in addition to the requirements in the following sections.

7.2.4 Borehole

The borehole shall be advanced to the rod anchor elevations into bedrock using suitable drilling techniques. The diameter of the borehole shall be sufficient to fit the rod, friction-reducing sleeve and rod anchor. The sides of the borehole shall be stable and the borehole shall be free of drilling mud and debris.

7.2.5 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.2.6 Rod Anchor

The rod shall be installed vertically in the borehole with its bottom end resting at the bottom of the borehole. The bottom portion of the rod shall be fixed against the surrounding bedrock by grouting the bottom 0.5 m of the borehole to form a concrete anchor.

Once grouting is completed and the rod anchor grout has set, the contractor shall pour clean sand in the lower 0.1 m length of the borehole above the concrete anchor to create a base for the end of the friction reducing sleeve to rest on.

The elevation of the bottom of the rod anchor shall be determined by measuring the length of the rod to the ground surface elevation.

7.2.7 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod above the rod anchor and sand, extending up to ground surface.

7.2.8 Installation Details

The elevation, easting and northing of the top of the Benchmark rod shall be surveyed.

The total distance from the rod anchor to the top of the rod shall be measured and recorded by the Contractor to an accuracy of ± 2 mm or better.

The Contractor is responsible for preventing damage to the benchmarks during the grade raise and surcharge construction. If a benchmark is damaged during fill placement, the rods, rod anchor, and friction-reducing sleeve shall be replaced before resuming the fill placement.

7.3 Settlement Plate (SP) Installation

7.3.1 General Procedure

The base plate of the settlement plate shall be installed on the top of the native subgrade. As grade raise and

surcharge construction proceeds, the rods shall be extended above the new top of grade raise fill and surcharge fill. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate.

7.3.2 Number and Location

The Contractor shall install SPs at the locations shown on the Contract Drawings and given in Table 1. The instrument locations should be field fit to avoid the Contractor's operations, but to be as close to the intended locations as practicable.

7.3.3 Installation

The Contractor shall install SPs as shown on the Contract Drawings and the Typical Monitoring Instrument Installation Details, in addition to the requirements in the following sections.

7.3.4 Plate

The settlement plate shall be installed horizontally on the top of a granular levelling pad constructed on the native subgrade following removal/stripping of any topsoil/organics.

7.3.5 Rod

The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

7.3.6 Friction-Reducing Sleeve

The friction-reducing sleeve shall be installed over the entire length of the rod that is below ground and within the grade raise and surcharge fill except that the cap on top of the SP rod shall extend 25 mm above the top of the friction sleeve at all times.

7.3.7 Extension of Rod

The SP rods shall be extended upwards as the grade raise and surcharge is constructed so that the top of the rod is always at least 0.3 m, but not more than 2 m above the surrounding fill.

7.3.8 Protective Surround

The CSP, friction-reducing sleeve and sand surround shall be extended concurrent with the rods, where applicable. The SP rod shall be in the centre of the CSP and friction-reducing sleeve. The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

7.4 Vibrating Wire Piezometer (VWP) Installation

7.4.1 General Procedure

The VWPs shall be installed in boreholes following removal/stripping of any topsoil/organics, and prior to grade raise and surcharge construction.

7.4.2 Number and Locations

The locations of the VWP's are shown on the Contract Documents and in Table 1. The VWP's shall be installed at the tip elevations shown on the Contract Drawings and in Table 1.

Installation of the VWP's shall be as per the manufacturer's recommendations in addition to what is stated or emphasized below.

7.4.3 Installation

The Contractor shall install VWP's as shown on the Contract Drawings and the Typical Monitoring Instrument Installation Details, in addition to the requirements in the following sections.

7.4.4 Borehole Installation

The borehole at each VWP location shall be advanced to approximately 900 mm above the target VWP tip elevation using suitable drilling techniques. The sides of the borehole shall be stable, and the borehole shall be free of drilling mud and debris. A split-spoon (SPT) sample (457 mm long) shall be taken at the bottom of the borehole to confirm the soil stratum above the VWP tip elevation after which the borehole shall be cleaned out to the bottom of the SPT sampling depth. The borehole shall be filled with water prior to installation of the VWP tip(s). The push-in VWP tip(s) shall be lowered to the bottom of the borehole and then pushed into the soil below the bottom of the borehole to the target VWP tip elevation.

7.4.5 Data Loggers

The data-logger(s) shall be installed in a protective casing at the VWP location. The protective casing shall be lockable, and weather proofed. The Contractor shall submit a detailed proposal of the protective casing to the Contract Administrator for review, prior to installation of the VWP's.

7.4.6 Completion of Installation

It is known that the process of installing VWP's can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VWP shall not be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily readings of the pore pressures, for the period noted below until the value has stabilized as determined by the Contract Administrator. Stabilization shall be deemed to have occurred:

- When no change in the measured value has occurred over a period of five (5) consecutive days and the measured value is within 10% of the anticipated hydrostatic value; and,
- When the daily rate of change is less than four (4) kPa per day for three (3) consecutive days and the measured value is within 5% of the anticipated hydrostatic value.

The Contractor should be prepared to wait for a period of 10 days to 15 days after completion of installation of the instruments for the baseline readings to stabilize.

7.5 Monitoring Program

7.5.1 Notification

The Contractor shall notify the Contract Administrator no later than three (3) working days after the completion

of installation of Benchmarks, Settlement Plates, and Vibrating Wire Piezometers.

7.5.2 Reporting

The Contractor shall supply the information outlined in the following sections to the Contract Administrator within three (3) days of completion of installation of each instrument.

7.5.2.1 Temporary Survey Benchmarks

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- TBM Northing and Easting in MTM NAD 83 coordinates
- Elevation of the rod anchor bottom, rod anchor length, and top of rod in Geodetic datum;
- Date of installation;
- Stratigraphic log of subsurface conditions at the TBMs, including notes on drilling method obstructions if encountered;
- Installation notes/sketches; and,
- Description of TBM (rod), sleeves and rod anchors.

7.5.2.2 Settlement Plates

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- SP Northing and Easting in MTM NAD 83 coordinates;
- Elevation of base of plate and top of rod and top of reinforcing bar in Geodetic datum;
- Date of installation;
- Installation notes/sketches; and,
- Description of SP rods, sleeves and plates.

Adjustments in the length of any SP rod shall be coordinated with the Contract Administrator to allow surveying by others of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

7.5.2.3 Vibrating Wire Piezometers

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- VWP Northings and Eastings in MTM NAD 83 coordinates;
- Elevations of VW sensors (tips) in Geodetic datum;
- Dates of installation;
- Stratigraphic log of subsurface conditions, including drilling method notes;
- Installation notes / sketches;
- Model, make and serial numbers of VW sensors, readout unit and signal cable; and,
- Calibration details of VW sensors.

7.5.3 Monitoring

7.5.3.1 Hand Over to CA and FES

The Contractor shall meet with the Contract Administrator and the Foundation Engineering Specialist (FES) responsible for the ongoing monitoring immediately after installation of the instruments and before the start of grade raise/surcharge construction. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments, and all equipment to be supplied by the Contractor, as identified in this special provision.

Monitoring by the Contract Administrator's representative (or FES) for the baseline readings shall commence within seven working days after the hand-over meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the construction of the grade raise/surcharge, and for up to approximately 7 months following the completion of the grade raise surcharge construction.

7.5.4 Decommissioning of Instruments

At the end of the monitoring period, the Contractor shall decommission all the temporary survey benchmarks and tie-in points by removing the rod and friction-reducing sleeve to at least 1.5 m below grade by excavating and backfilling with compacted granular fill in accordance with the specifications for fill placement.

At the end of the monitoring period, the Contractor shall decommission all Settlement Plates, unless otherwise advised by the Contract Administrator. The Vibrating Wire Piezometers shall remain in place and in operation. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources Act, Regulation 903 (as amended).

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.1 Supply and Installation of Grade Raise and Surcharge Monitoring Equipment - Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work, including the supply, installation and decommissioning of survey benchmarks, Settlement Plates, and Vibrating Wire Piezometers, as well as performing all required monitoring and reporting.

FOUNDATION ENGINEERING SPECIALIST FOR FOUNDATION MONITORING PROGRAM
– Contract 2024–4033

Terms of Reference for Foundation Engineering Specialist for Monitoring for CA Assignment

General

The purpose of this Foundation Monitoring Program is to monitor settlements and pore water pressures in the foundation soils at select locations during construction of the grade raise and surcharge within the footprint of the Vehicle Maintenance and administration Building (VMB) of the Arnprior Maintenance Yard as described in the contract documents.

The instrumentation monitoring services shall include: data collection; data reduction and reporting; adherence to criteria used to assess the post-construction behaviour of the grade raise and surcharge fill based on the monitoring data collected from the instruments installed by others.

The Foundation Engineering Specialist (FES) shall:

- Meet with the Contractor to receive the logger manager software and cabling used for downloading data from the Dataloggers that are monitoring the vibrating wire piezometers, and to receive reports with details about installation of instruments installed by the Contractor and calibration certificates, as specified in the Special Provision titled, “Supply and Installation of Embankment Monitoring Equipment”, included in the contract documents;
- Take instrument readings, reduce data, prepare reports;
- Interpret instrumentation readings as needed for the purpose of on-going construction and for providing geotechnical input to the completion of the surcharge period (note: a minimum degree of consolidation of 70% within the silty clay stratum is required at completion of the surcharge period); and,
- Notify the Contract Administrator (CA) if critical instrument readings (Review and Alert Levels) are reached. Discuss as soon as possible (within 48 hours) with the Contract Administrator response action(s), and submit a plan of actions, to prevent the instrument readings from exceeding the critical levels.

Progress Reports shall be submitted to the Contract Administrator weekly during the grade raise filling and surcharge construction and monthly until surcharge removal. The Progress Reports shall discuss the Contractor's operations with respect to the installation of instrumentation, extent of fill placed and summary of monitoring completed and data collected.

At the completion of the monitoring program, a Final Monitoring Report shall be issued to the Contract Administrator. The monitoring results shall be presented in tabular and graphical form. Interpretation of the monitoring data shall be included in the report.

The Foundation Engineering Specialist shall save and archive the raw monitoring data in electronic and hard copy format.

Specialist Qualifications

The Foundation Engineering Specialist (FES) Consultant services required for this assignment have been categorized Geotechnical specialty – High Complexity.

The FES Consultants that are registered in MTO's consultant acquisition system (RAQS) at complexity ratings in the required specialty that meet or exceed the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Monitoring Consultant shall not be the same Geotechnical Consultant retained by the Contractor for the supply and installation of the monitoring equipment.

Settlement Plate (SP) and Vibrating Wire Piezometer (VWP) Monitoring

The Foundation Engineering Specialist shall carry out instrumentation monitoring for the following items:

- Settlement Plates (SPs); and,
- Vibrating Wire Piezometers (VWPs)

The Settlement Plates (SPs) and Vibrating Wire Piezometers (VWPs), as well as the Temporary Benchmark (TBM) shall be supplied and installed by the Contractor. The quantities and locations of the settlement plates and vibrating wire piezometers are shown in the Table 1.

Table 1: Instrument Quantities and Locations

Location	SP	VWP	TMB
Arnprior Maintenance Yard (Vehicle Maintenance and administration Building, VMB)	4	2	1

Reading Schedule and Frequency

Monitoring shall commence immediately after the installation of an instrument. Monitoring is to continue during a period from the start of embankment construction until surcharge removal. The actual length of the monitoring period depends on the construction schedule.

The minimum monitoring frequencies along with the anticipated number of readings is provided in Table 2. The monitoring frequency is the same for each individual instrument. Instruments shall be read more or less frequently if judged to be required by the Foundation Specialist. It should be noted that the number of readings provided in Table 2 are estimates and may vary depending on the actual construction schedule.

Table 2: Minimum Monitoring Frequency

STAGE	FREQUENCY ²	ANTICIPATED NUMBER OF READINGS PER INSTRUMENT
Baseline Readings ¹	Daily for 3 consecutive days prior to start of fill placement, no sooner than 3 days following installation.	3
Immediately prior to start of fill placement	Once	1
During fill placement for grade raise and surcharge construction	Daily	10 ³
During surcharge period (i.e., following completion of grade raise and surcharge construction)	Once every week for the first 2 months	8
	Once every two weeks from 2 months to 4 months	4

STAGE	FREQUENCY ²	ANTICIPATED NUMBER OF READINGS PER INSTRUMENT
	Once every month from 4 months to 7 months	3

Note(s):

1. Baseline Readings: Value of instrumentation readings taken prior to start of construction/fill placement to provide baseline against which all subsequent readings are compared to assess movements of ground.
2. VWP data is collected by dataloggers 24 times a day (i.e., one reading every 1 hour). For the VWPs, the 'Frequency' indicates the number of times data from the VWPs is to be downloaded from dataloggers.
3. Due to uncertainty in the construction schedule, the number of readings may be greater than indicated.

Settlement Plates (SP)

The elevations of Settlement Plates shall be surveyed to an accuracy of plus/minus two (± 2) mm or better. Surveying for shall be conducted by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the CA.

A full set of up-to-date and processed survey monitoring data shall be presented in tabular and graphical form in the Progress Reports. As a minimum, the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the SPs:

- A plot of settlement of the base of the fill at the Settlement Plates (SPs) versus time;
- Fill height within 5 m of the instruments versus time;
- Plan view, cross section and profile sketches showing the approximate limits of fill placement while the SPs were being surveyed; and,
- All plots should be presented in both linear time and log time.

Typically, embankment (fill) failures result in an acceleration of settlements after placement of a lift of fill. If this condition is observed or the maximum settlement measured exceeds the Review Levels in Table 3, the FES shall immediately inform the CA and discuss response action(s). The FES shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that instrument Alert Levels are not reached.

If the maximum settlement measured exceeds the Alert Levels in Table 3, the FES shall immediately inform the Contract Administrator who shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

- The cause of the accelerated settlement has been identified and analyzed by the FES;
- Any corrective action deemed necessary by the FES has been implemented; and,
- The Contract Administrator deems it safe to proceed.

Table 3: Review and Alert Levels for Settlement Plates (SPs)

Location		Name	Settlement Response Levels (mm)	
Northing (m)	Easting (m)		Review	Alert
5030649.994	318064.093	SP24-01	50	75
5030668.575	318084.844	SP24-02	50	75
5030653.398	318098.527	SP24-03	50	75
5030634.596	318077.799	SP24-04	50	75

Vibrating Wire Piezometers (VWP)

The VWPs shall be read using the Logger Manager software and cabling used for downloading data from the Dataloggers supplied by the Contractor.

A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical form in the Progress Reports. As a minimum, the following shall be submitted to the Contract Administrator in the Progress Reports based on the readings collected from the VWPs:

- A plot of piezometric head (elevation), background piezometer head (elevation) and fill elevation versus time for each VWP;
- A plot of excess pore pressure (EPP) and embankment (i.e., grade raise+surchage) vertical effective stress versus time for each VWP;
- Plan view, cross section and profile sketches showing the approximate limits of fill placement at the time the VWP readings were downloaded; and,
- Review and Alert levels on EPP versus time plots.

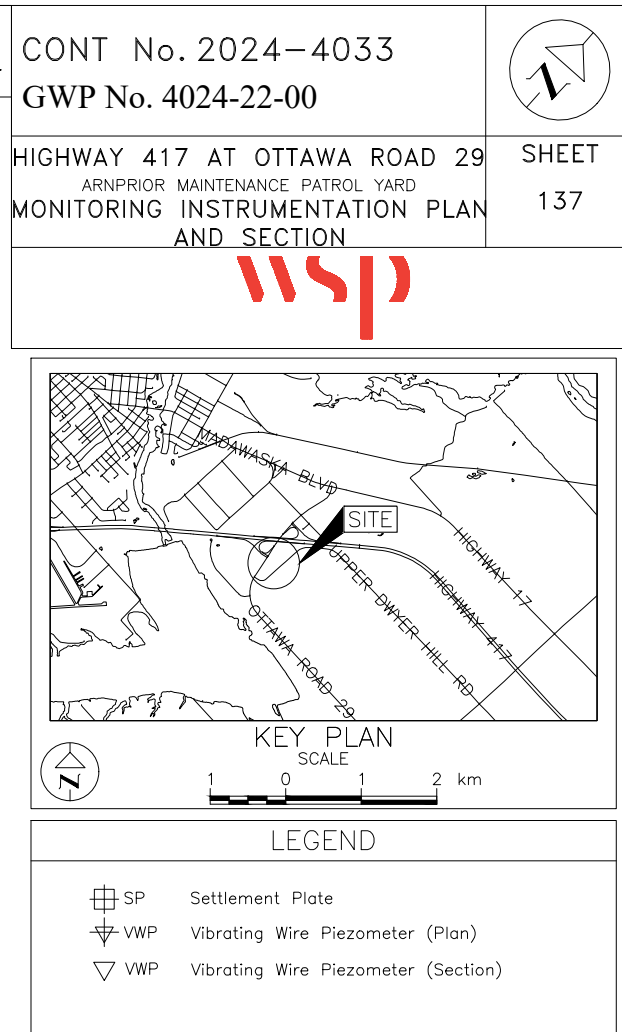
The increase in pore pressure in the foundation soils associated with the placement of successive lifts of fill should be equal to or lower than the increase in total vertical stress due to the fill placement. The failure of embankments founded on soft soils is usually associated with increases in pore pressure to levels which exceed the increase in total stress as described above. If this condition is observed or the maximum excess pore pressure measured exceeds the Review Levels in Table 4, the FES shall immediately inform the CA and discuss response action(s). The FES shall submit a plan of action(s) to prevent Alert Levels being reached. All construction work shall be continued such that the instrument Alert Levels are not reached.

If the maximum excess pore pressure measured exceeds the Alert Levels in Table 4, the FES shall immediately inform the CA who shall instruct the Contractor to stop all construction activities on and within the embankment until all the following conditions are satisfied:

- The cause of the excess pore pressure has been identified and analyzed by the FES;
- Any corrective action deemed necessary by the FES has been implemented; and,
- The Contract Administrator deems it safe to proceed.

Table 4: Review and Alert Levels

Location		Name	Excess Pore Pressure (EPP) - Response Levels (kPa)	
Northing (m)	Easting (m)		Review	Alert
5030659.253	318074.434	VW24-01	50	70
5030648.986	318083.627	VW24-02	50	70




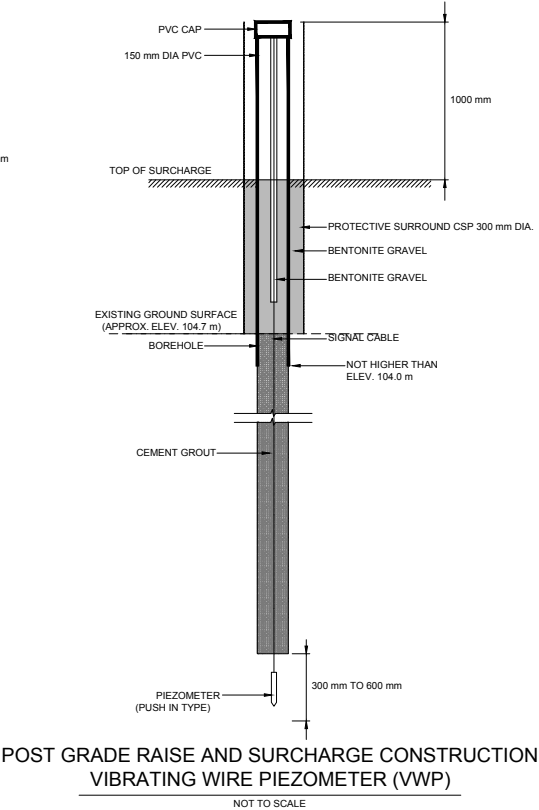
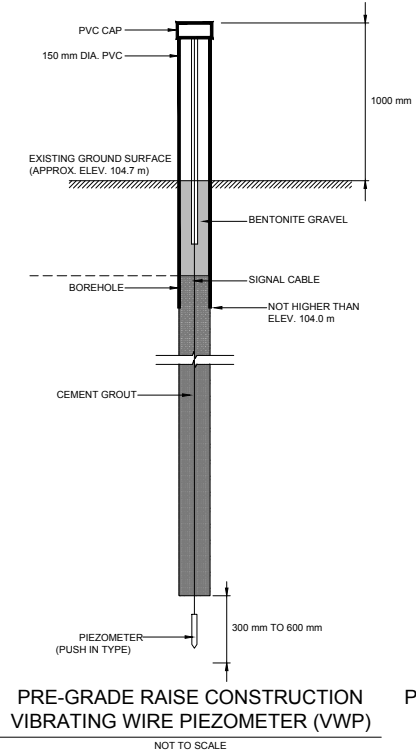
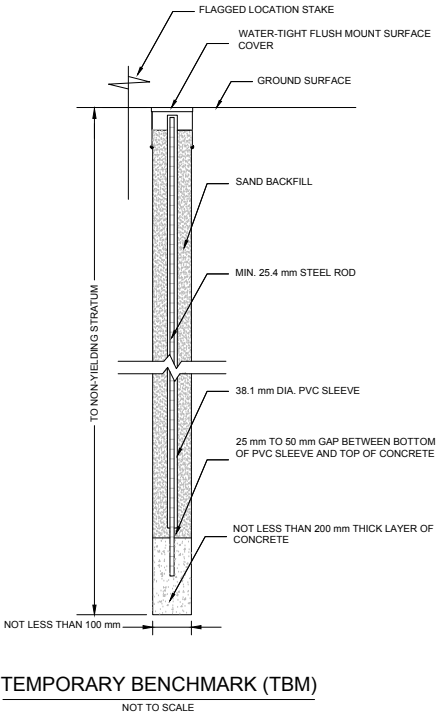
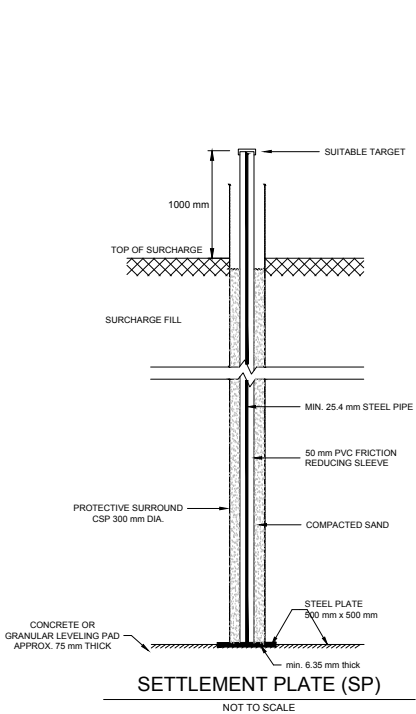
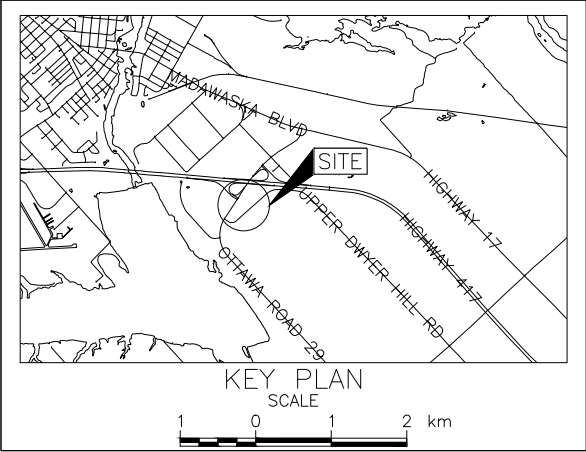
NO.		DATE		BY		REVISION	
Geocres No. 31F08-001							
HWY. 417				PROJECT NO. CA0012298_456 DIST. EASTERN			
SUBM'D. KCP		CHKD. KCP		DATE: 5/29/2024		SITE:	
DRAWN: JM/SA		CHKD. KCP		APPD. DS		DWG. 4	

METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2024-4033
GWP No. 4024-22-00

HIGHWAY 417 AT OTTAWA ROAD 29
TYPICAL MONITORING INSTRUMENT INSTALLATION
DETAILS


SHEET
138



NOTES
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.



NO.	DATE	BY	REVISION
Geocres No. 31F08-001			
HWY. 417		PROJECT NO. CA0012298_4565	
SUBM'D. KCP		DIST. EASTERN	
DRAWN: DD/SA		SITE: .	
CHKD. KCP		DWG. 5	
APPD. DS			

