FINAL REPORT

Preliminary Foundation Investigation and Design Highway 401 / Power Dam Drive Underpass Replacement

GWP 4092-19-00

Submitted to:

Morrison Hershfield Ltd.

2932 Baseline Rd,
Nepean, ON
K2H 1B1

Submitted by:

Golder Associates Ltd.

1931 Robertson Road, Ottawa,
Ontario, K2H 5B7

Latitude 45.058740°

Longitude: -74.808730°

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PART A

Preliminary Foundation Investigation Highway 401 / Power Dam Drive Underpass Replacement GWP 4092-19-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the Highway 401 / Power Dam Drive underpass structure as part of the Mega 18 Project (Purchase Order No. 4019-E-0023; GWP 4092-19-00).

This report presents the results of the foundation investigation carried out near each of the abutments of the existing underpass. The purpose of this preliminary foundation investigation was to assess the subsurface conditions at the existing abutments, and to provide geotechnical input for provide preliminary design recommendations for the replacement of the Highway 401/Power Dam Drive underpass structure. The foundation investigation included drilling boreholes, installing a groundwater monitoring well and subsequent laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in MTO's Work Item Order Form for Assignment 4, dated September 4, 2020. Golder's scope of work for the preliminary foundation engineering services associated for this project underpass was provided in the Work Order for this assignment dated October 8, 2021

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The existing underpass, which carries two lanes of Power Dam Drive over Highway 401, is located about 5 km west of the city of Cornwall within the county of South Stormont, Ontario. The location of the underpass structure is shown on the Key Plan on Drawing 1. Site photographs showing the general conditions at the site are presented in Appendix D.

Power Dam Drive is a divided road with a single travel lane in each direction separated by a narrow concrete curb and asphalt median. Steel cable guiderails are present along both side of the Power Dam Drive in the vicinity of the underpass structure.

At this location, Highway 401 has a four-lane cross-section with two eastbound and two westbound through lanes separated by a wide, vegetated median. Steel beam guiderails are also present along both sides of the highway in the vicinity of the underpass structure. The interchange includes a westbound onramp and an eastbound offramp from/to Power Dam Drive.

No existing construction drawings or GEOCRES reports were available for the existing structure at the time this report was prepared.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*[1], this section of Highway 401 lies within the minor physiographic region known as the Glengarry Till Plain, which lies within the major physiographic region known as the Ottawa-St. Lawrence Lowland.

1 Chapman, L. J. and Putnam, D. F., 1984. The Physiography of Southern Ontario, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

The Glengarry Till Plain region is characterized as lowlands in which the surface is undulating to rolling, consisting of long morainic ridges and a few well-formed drumlins together with intervening clay flats. The deposit of sand and gravel till is very stony and contains large near surface boulders.

There is no available GEOCRES information at the Power Dam Drive interchange, but, based on the 1998 General Arrangement drawing (prepared in 1998 for a planned rehabilitation of the bridge), the existing north bridge abutment and piers are supported on shallow foundations on rock and the south abutment is perched within the existing embankment supported on piles driven to bedrock. A copy of the 1998 General Arrangement drawing is provided after the text of this report. It should be noted that no elevation data is provided on this drawing and that no further information about the construction of the existing structure was available at the time of preparation of this report.

From a foundation perspective, this site is geologically complex consisting of moraines of sandy till rising through deposits of marine clay. The soil conditions approximately 2 km west of the interchange, at Post Road and Highway 401, consist of sand and silt till over bedrock while, just 1 km west of the site at Culvert 447, a relatively thick (7 to 10 m) layer of soft, compressible clay was encountered beneath the surficial sand. At the southwest, northwest and northeast quadrants of the interchange, sandy till is indicated, however, a 12 m thick deposit of clay is indicated to exist in the northeast quadrant. Between 0.85 and 1 km east of the site, the subsurface conditions seem to generally consist of till with thin interbedded layers of silty clay over bedrock.

The site falls within the Western Québec (WQ) seismic zone according to the Geological Survey of Canada. The WQ zone constitutes a large area which encompasses the urban areas of Montreal, Ottawa-Hull and Cornwall. Within the WQ zone recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. The two major earthquakes that have recently occurred in the WQ zone are the 1935 Témiscaming event, which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2, and the 1944 Cornwall-Massena event, which had a magnitude of 5.6.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The fieldwork for the current investigation was carried out on October 27 and October 28, 2021, and included advancing 2 boreholes, numbered 21-01 and 21-02. The boreholes were located within the Highway 401 right-of-way at the existing north and south abutments.

The boreholes were advanced using truck mounted drilling equipment supplied and operated by George Downing Estate Drilling Limited of Hawkesbury, Ontario.

Traffic control required to close the driving lanes and shoulders of the highway while carrying out the field operations was provided by Beacon Lite Limited of Ottawa, Ontario.

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.

Bedrock core samples were obtained using NQ3 diamond drilling equipment in both boreholes.

A monitoring well was installed in Borehole 21-02, to observe the stabilised groundwater level at the site. The monitoring well consists of a 32 mm outside diameter PVC tubing with a 1.52 m long slotted screen section. The groundwater level was measured in the well on November 4, 2021, and June 7, 2022. The well was subsequently decommissioned according to Ontario MOE Regulation 903 (O.Reg 903) by a licenced well technician.

Borehole 21-01 was backfilled with bentonite within the bedrock, and bentonite mixed with soil cuttings within the overburden in general accordance with the intent of O.Reg 903, as amended. The borehole was then capped with granular material to match the surrounding surface cover. The site conditions were restored following completion of the fieldwork.

The fieldwork was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, logged the boreholes and examined and cared for the soil and bedrock samples retrieved. The soil and bedrock samples were identified in the field, placed in labelled containers, and transported to Golder'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. Uniaxial Compressive Strength (UCS) tests were carried out on selected rock core samples by Stantec Consulting Ltd. The laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate at Golder's Ottawa laboratory.

Classification of the rock mass quality of the bedrock core samples with respect to the Rock Quality Designation (RQD) and UCS are described based on Table 3.10 and Table 3.5, respectively, of the *Canadian Foundation Engineering Manual* (CFEM, 2006[2]). The degree of weathering of the bedrock samples and the strength classification of the intact rock mass based on field identification are described in accordance with Table B.3 and Table B.6, respectively, of the International Society for Rock Mechanics (ISRM)[3] standard classification system.

One soil sample was 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 Golder using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 8 geodetic datum. All elevations are geodetic referencing CGVD 1928 using Geoid Model HT2_0e. The borehole locations, including northing and easting coordinates, CR43 Stationing, ground surface elevations, and drilled depths are summarized in Table 1.

Table 1: Borehole Location Summary

Borehole	Borehole Location	Station	NAD83 CSRS CBNv62010.0 MTM Zone 8		Ground Surface Elevation (m)	Drilled Length (m)
			Northing (m)	Easting (m)		
21-01	HWY 401 eastbound lane	18+820	4991806.3	201726.8	64.4	8.2
21-02	HWY 401 westbound lane	18+880	4991876.9	201689.7	64.4	7.7

2 Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

3 International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The subsurface soil, and groundwater conditions encountered in the boreholes and the results of in-situ testing from the current investigation are given on the Record of Boreholes, 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 on Figures B1 to B4 in Appendix B. The borehole locations and the interpreted stratigraphic profiles projected along Power Dam Drive are provided on Drawing 1.

4.2 Site Stratigraphy Overview

At the boreholes, the subsurface conditions generally consist of granular surface cover, overlying fill materials, overlying a very stiff weathered clay crust overlying a firm to stiff clay, which in turn overlies a gravelly silty sand glacial till, all underlain by limestone bedrock. The final stabilized groundwater level was measured in Borehole 22-02 on June 7, 2022, at a depth of 1.4 m, corresponding to Elevation 63.0 m.

The stratigraphic boundaries shown on the Record of Borehole and Drillhole sheets and on the interpreted stratigraphic section on Drawing 1, are inferred from observations of 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.

4.3 Surface Cover / Surficial Materials

A layer of topsoil was encountered at the ground surface at Borehole 21-02 with a thickness of about 50 mm.

4.4 Fill

Boreholes 21-01 and 21-02 were advanced through the right shoulder of the eastbound and westbound lanes of Highway 401, respectively. Fill consisting predominantly of gravelly silty sand was encountered at the ground surface at Boreholes 21-01 and below the topsoil at Borehole 21-02. The thickness of the of the fill layer ranges between 1.2 to 1.7 m. The SPT N values measured within this deposit ranged from 11 to 28, indicating a compact state of compactness. The measured moisture content of a single sample of the sand fill material tested was 12%. The results of grain size analysis testing carried out on a single sample of this material are provided on Figure B1 in Appendix B.

4.5 Clay

A clay deposit was encountered beneath the fill layer at both boreholes.

The upper portion of the deposit has been weathered to a stiff crust. The top of the clay crust was encountered at elevations ranging from 62.7 to 63.2 m. The thickness of the clay crust ranges from 1.4 to 1.9 m. The SPT N values ranged from 6 to 11 blows per 0.3 m of penetration, indicating a very stiff consistency.

The moisture content of the one sample of the clay crust tested was 42%. The results of grain size analysis testing carried out on one sample of this material are illustrated on Figure B2 in Appendix B. The results of Atterberg Limits testing completed on a single sample of the weathered crust indicate a liquid limit of 89, a plastic limit of 28 and plasticity index of 61. The Atterberg Limits analysis results are illustrated on Figure B3 in Appendix B and indicate a clay of high plasticity (CH).

The clay below the depth of weathering is grey. The top of the grey clay was encountered at Elevation 61.4 m in both boreholes. The thickness of the grey clay is 0.7 and 0.9 m. SPT N values ranged from weight of hammer to 2 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

The moisture content of the two samples of the grey clay tested were 65 and 69%. The results of grain size analysis testing carried out on two samples of this material are illustrated on Figure B2 in Appendix B. The results of Atterberg Limits testing completed on two samples of the grey clay indicate liquid limits of 70 and 75, plastic limits of 23 and 27 and plasticity indices of 47 and 48. The Atterberg Limits analysis results are illustrated on Figure B3 in Appendix B and indicate a clay of high plasticity (CH).

The calculated liquidity indices vary from 0.8 to 1.0, indicating the measured natural water content of the selected samples is generally at or below their liquid limit values.

4.6 Glacial Till

Glacial till was encountered below the grey clay in both boreholes. The glacial till generally consists of a soil matrix of gravelly sandy clayey silt. The top of the glacial till was encountered at Elevations 60.5 to 60.7 m at Boreholes 21-01 and 21-02 respectively.

The SPT N values ranged from 9 to greater than 50 blows per 0.3 m of penetration indicating stiff to hard. The higher blow count (i.e., 50/150mm) recorded at Borehole 21-01 for the till may have been influenced by the presence of cobbles or boulders within the till or the presence of bedrock, rather than the consistency of the soil matrix.

The measured moisture content of the one sample tested was 21%. The results of grain size analysis testing carried out on a single sample of the glacial till are provided on Figure B4 in Appendix B.

4.7 Bedrock

The overburden materials are underlain by limestone bedrock with shale partings and interbeds.

Bedrock core samples were obtained using NQ3 sized diamond drilling equipment at both boreholes.

Table 2 summarizes the depths and the elevations of the bedrock surface as encountered at the borehole locations.

Table 2: Summary of Bedrock Surface Depths and Elevations

Borehole	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
21-01	64.4	4.3	60.1
21-02	64.4	4.5	59.9

The Total Core Recovery measured on the core samples ranges from 95 to 100%. RQD values generally ranged from about 94 to 100%, indicating a rock mass of excellent quality.

UCS testing was carried out on two bedrock core samples. The samples tested had UCS values of 127 and 109 MPa indicating a very strong bedrock.

4.8 Groundwater Conditions

A monitoring well was installed in Borehole 21-02, to observe the stabilized groundwater level at the site.

Table 3 summarizes the depths and the elevations of the groundwater levels measured at the boreholes and piezometers installed at the site

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

Table 3: Summary of Groundwater Conditions

Borehole	Type	Screened Interval	Ground Surface Elevation (m)	Depth (m)	Elevation (m)	Date
22-02	Piezometer	Clay / Glacial Till	64.4	1.1	63.3	November 4, 2021
				1.4	63.0	June 7, 2022
22-01	Open borehole	-	64.4	2.5	61.9	October 27, 2021

4.9 Steel Corrosion and Sulphate Attack, Chemical Analysis

One soil sample was submitted to Eurofins 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 and are summarized in Table 4.

Table 4: Steel Corrosion and Sulphate Attack, Chemical Analysis

Borehole	Sample	Sample Elevation (m)	Chloride (%)	Sulphate (%)	Electrical Conductivity (mS/cm)	pH	Resistivity (ohm-cm)
21-02	SS3	1.8	<0.002	0.01	1.36	7.8	741

5.0 CLOSURE

This report was prepared by Ben Waechter, EIT, and Kenton Power, P.Eng. The report was reviewed by William Cavers, P.Eng. an Associate, Senior Geotechnical Engineer with Golder and the Designated MTO Foundations Contact for this project.

Golder Associates Ltd.

Ben Waechter, EIT
Geotechnical Engineer in Training

LICENSED PROFESSIONAL ENGINEER
K. T. C. POWER
100130652
June 9, 2022
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
W. CAVERS
100077934
09-06-2022
PROVINCE OF ONTARIO

Kenton C. Power, P.Eng.
Geotechnical Engineer

William Cavers, P.Eng
Designated MTO Foundations Contact

BW/KCP/WC/hwd

https://golderassociates.sharepoint.com/sites/144785/project files/6 deliverables/3-final/21464403-rev0-fidr final hwy 401-power dam 2022-06-03.docx

PART B

Preliminary Foundation Design Highway 401 / Power Dam Drive Underpass Replacement GWP 4092-19-000

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary geotechnical input for the for the replacement of the Highway 401 / Power Dam Drive underpass structure as part of the Mega 18 Project (Purchase Order No. 4019-E-0023; GWP 4092-19-00). The input provided herein is based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation, and in accordance with the current Canadian Highway Bridge Design Code CSA S6:19 (CHBDC).

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 Existing Conditions

Power Dam Drive, within the area under consideration, is a divided road with a single travel lane in each direction separated by a narrow concrete curb and asphalt median. At this location Highway 401 has a four-lane cross-section with two eastbound and two westbound through lanes separated by a wide, vegetated median. Steel cable guiderails are present along both side of Power Dam Dive and Highway 401 in the vicinity of the underpass structure.

No construction drawings or GEOCRES reports were available for the existing structure at the time of this report was prepared.

The base plan mapping provided by MH for this project and the ground surface elevations at the borehole locations surveyed during the current field investigation indicate that the top of roadway elevation of Highway 401 in the vicinity existing structure is at approximately Elevation 64.4 m. No boreholes were advanced through the existing approach embankment during this preliminary investigation. However, based on visual inspection at the time of the field investigation the approach embankments are approximately 5 to 7 m in height above the natural ground level and were constructed with side slopes that are generally oriented at about 2 horizontal to 1 vertical (2H:1V).

6.2 Seismic Design

6.2.1 Seismic Hazard and Importance Category

Section 4.4.3 of the CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, it is understood that Highway 401 at this location has been given an importance category of “Major Route”.

6.2.2 Seismic Site Classification

In accordance with the Table 4.1 of the CHBDC, 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. As described in Section 6.3.2 the frost penetration depth at this location is 1.7 m. At the time of preparing this preliminary report the founding elevations of the proposed abutments has yet to be determined and therefore the frost penetration depth at the site of 1.7 m below the existing ground surface has been assumed as the minimum founding depth for this project.

Based on the current understanding of the foundation conditions at the site (i.e., strip footings founded greater than 3.0 m above the limestone bedrock), the site would be classified as a Seismic Site Class D in accordance with Table 4.1 of the CHBDC. It may be possible to upgrade the classification from a Site Class D to a more favourable Site Class C if shear wave velocity testing is carried out at the proposed site. Further discussion is provided in *Additional Investigation Works* Section below.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the Highway 401 / Power Dam Drive interchange (latitude 45.059N; longitude 74.809W), the values provided in Table 5 are the reference Site Class C (reference) peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

Table 5: Site Class C Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year) (g)
PGA	0.375
T <= 0.2 s	0.589
T = 0.5 s	0.308
T = 1.0 s	0.146
T = 2.0 s	0.067
T = 5.0 s	0.017
T => 10.0 s	0.006

The values given in Table 5 are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given in Section 6.2.2 (Site Class D) in accordance with Section 4.4.3 of the CHBDC. As indicated in Section 4.4.3.3 of the CHBDC, the value of PGA_{ref} for use with Tables 4.2 to 4.9 shall be taken as 80% of the PGA for Site Class C where Sa (0.2)/PGA is less than 2.0. Based on this requirement a PGA_{ref} value of 0.3g was used for the 2,475-year return period. The corresponding site-specific Site Class D seismic hazard values given in Table 6 can be used for design.

Table 6: Site Class D Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year) (g)
PGA	0.371
T <= 0.2 s	0.589
T = 0.5 s	0.370
T = 1.0 s	0.191
T = 2.0 s	0.091
T = 5.0 s	0.024
T => 10.0 s	0.008

The fundamental period of the replacement structures has yet to be confirmed and may depend on the final design of the superstructure. In consideration of the structure's "Major Route" importance category and the site specific seismic hazard values given in Table 6, in accordance Table 4.10 of the CHBDC the bridge would fall in Seismic Performance Category 2, if the fundamental period of the structure is greater than or equal to 0.5 s, or Seismic Performance Category 3, if the fundamental period of the structure is less than 0.5 s.

6.3 Foundation Options

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the existing underpass structure and foundation systems may be classified as having medium to large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding" for these sites. Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ of 1.0, and geotechnical resistance factors from Table 6.2 of the CHBDC have been used for design, as indicated in the following sections.

As per Section 6.14.4 of the CHBDC for seismic design the consequence factor, Ψ, should be taken as 1.0 while the resistance factor, ϕ_{gu}, should be taken from Table 6.3 based on the structural design approach.

6.3.2 Frost Protection

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Penetration Depths for Southern Ontario), the frost penetration depth at the site is 1.7 m below the existing ground surface. Footings constructed at this site or the underside of pile caps should have a minimum embedment depth of 1.7 m for frost protection purposes.

6.3.3 Foundation Design Alternatives

The results of the field and laboratory investigation indicate that the site soil stratigraphy consists of fill material overlying a relatively thin layer of clay, overlying a thin layer of glacial till all underlain by limestone bedrock.

Key elevations are as follows:

- Existing ground surface elevation of Highway 401 is approximately Elevation 64.4 m
- Top of the clay crust is at Elevations 63.2 and 62.7 m
- Top of glacial till deposit is at Elevations 60.5 and 60.7 m
- Top of bedrock surface is at Elevations 60.1 and 59.9 m

The clay can generally be characterized as moderately sensitive with high plasticity. The clay below the weathered crust is generally firm to stiff and offers low bearing resistance and is susceptible to settlement under even moderate loads. As such the clay deposit has insufficient strength to support the anticipated foundation loads associated with the proposed abutments and piers.

The glacial till deposit generally consists of a loose, thin layer (i.e., less than 1.0 m in thickness) of gravelly sandy clayey silt with containing cobbles and boulders. Based on the results of the SPT N values measured in the till, the deposit has insufficient strength to support the anticipated foundation loads. Also, the relatively deep excavations required to reach the till surface also make founding on the till likely not practical from a constructability standpoint.

Based on the anticipated loading it is considered feasible to found spread footings on the limestone bedrock. However, excavations upwards of 5 m of would be required to reach the bedrock surface to construct the footings. Challenging and extensive dewatering requirements should be anticipated at this site should deep excavations be carried out (i.e., possible artesian groundwater conditions in the till and bedrock). Although the bedrock could support the abutment loading it is likely not practical to excavate to the required depth to found spread footings and construct the abutments at this site.

Based on the foregoing, shallow foundations are not considered a feasible foundation option as the fills and native materials would not provide sufficient bearing capacity to support the bridge loading and therefore the footings would need to be founded on the limestone bedrock. Relatively deep excavations below the groundwater would also be required to found the footings on the bedrock.

Therefore, deep foundations, founded on or in the bedrock, are anticipated for this site. Relatively short piles will be required as the depth to the bedrock surface is between 4.3 and 4.6 m below the top of pavement elevation of Highway 401.

From a geotechnical perspective, the subsurface conditions at the site are generally suitable for integral or semi-integral abutments perched within the approach fills.

Based on the existing General Arrangement Drawing and the frost penetration depth at the site of 1.7 m an underside of the abutment pile cap elevation of 65.0 m has been assumed for the following evaluation of deep foundation options.

As part of the review of foundation alternatives driven steel H-piles, drilled-in pipe piles (down-the-hole hammer) and, augered Concrete Caissons (drilled shaft piles) were considered.

A comparison of foundation alternatives, including advantages, disadvantages, risks and relative costs is provided in Table 11 following the text of this report.

1) Steel H-piles are considered feasible at this site to support the foundation elements. With bedrock between 4.3 and 4.7 m below the top of the pavement surface of Highway 401, relatively short piles are anticipated. As such socketing the piles in bedrock will likely be required to provide sufficient lateral and uplift support for the structure. Socketing of the piles into the bedrock would also be required for integral abutments and the sockets would need to be deeper, with the upper part of the rock socket ungrouted to allow sufficient flexibility for abutment movements. This requirement may be avoided by perching integral abutments within the embankment fill. For H-piles socketed into the bedrock (if required), pre-drilling holes, likely requiring the mobilization of two separate pieces of equipment and increased installation time due to the multi-step installation process, would be necessary. Based on the foregoing, although H-piles are considered to be feasible at this location from a foundations perspective, they may not be practical from a construction and cost perspective, unless integral abutments, ideally perched within the embankment fill, are desired.

2) The foundations elements could also be founded on drilled-in steel HSS pipe piles socketed into the bedrock installed using a down-hole hammer installation method. The advantage is that a single piece of equipment could be used to advanced through the overburden and create the bedrock socket without the need to pre-drill the holes. However, there are a smaller number of contractors with suitable equipment. Not commonly used for integral abutments and could require non-traditional detailing which would need to be confirmed by structural designer.

3) Concrete caissons founded on or in the bedrock could also be considered for support of the new structure however existing site conditions present difficulties for installation of caissons. The caissons will need to be installed below the groundwater level and temporary steel casings would therefore be required to keep the hole open during drilling. Depending on the length of the caisson it can be difficult to clean and inspect the base prior to placing concrete. Caissons are also not compatible with an integral abutment design approach. Based on the foregoing, caissons are not considered as a feasible option at this site from a foundations perspective and are not considered further in this report.

6.3.4 Recommended Foundation

Based on the evaluation of foundation alternatives presented above, the preferred foundation approach from a foundations perspective is to support the founding elements on drilled-in steel pipe piles socketed into the bedrock.

Steel H-piles installed in pre-drilled holes socketed into bedrock are also considered feasible from a foundation perspective.

6.4 Deep Foundations

6.4.1 General

As the design of the structure has yet to be finalized the following two standard piles sizes have been used in the following foundation assessment. Further assessment will be required during the detailed design stage when loading and founding elevation are known.

- 324 x 12.7 mm steel pipe pile; and,
- 310 x 110 steel H-pile

The abutments may be founded on either steel H-piles or HSS pipe piles socketed a minimum of 0.5 m into sound bedrock; for integral abutments the sockets will need to extend at least 5 metres below the underside of pile cap. Bedrock sockets may not be required for integral abutments, if they are perched within the embankment fill, provided there is sufficient lateral and uplift resistance. The estimated pile tip elevations, the underside of pile caps are 1.7 m below ground surface and the piles are nominally socketed. are summarized in Table 7.

Table 7: Estimated Pile Tip Elevations

Foundation Element	Relevant Borehole	Approximate Underside of Pile Cap Elevation[1] (m)	Bedrock Socket Length (m)	Estimated Pile Tip Elevation (m)	Estimated Pile Length (m)
North Abutment	21-02	65.0	0.5	59.4	5.6
South Abutment	21-01			59.6	5.4

Note: [1] Underside of footing elevation assumed based on existing General Arrangement and frost penetration dept

Piles must be installed in accordance with OPSS.PROV 903. As per Section 903.07.03.01 of OPSS.PROV 903 the contract documents should indicate that the piles should be advanced into bedrock with a socket length of 0.5 m (depending on the underside of pile cap elevation).

6.4.2 Preliminary Factored Geotechnical Axial Resistance

Based on the estimated uniaxial compressive strength of the rock at this site and assuming good to excellent rock quality, the following factored axial geotechnical resistance can be used in the design.

- 324 x 12.7 mm steel pipe pile; factored geotechnical resistance at ULS of 6,500 kN per pile; and,
- 310x110 steel H-pile factored geotechnical resistance at ULS of 8,000 kN per pile.

If required, the piles could be socketed into the bedrock with a nominal socket length of 0.5 m. 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 bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Assuming an importance category of “Major Route” bridge in accordance with Section 4.4.2 of the CHBDC and a typical degree of site and prediction model understanding as per Section 6.5 of the CHBDC, a geotechnical resistance factor of ϕ_{gu} = 0.4 from Table 6.2 of the CHBDC has been used to calculate the factored geotechnical resistance values provided.

6.4.3 Resistance to Lateral Loads

It is understood that lateral loading will be resisted partially by the steel piles at the abutments. Additional resistance to lateral loading may be derived from the soil in front of the piles.

The ULS geotechnical resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the Commentary to the CHBDC, assuming that it acts over the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and an equivalent width equal to three pile diameters.

For a 324 x 12.7 mm steel pipe pile the unfactored ULS lateral resistance is 400 kN per pile at the abutment locations.

For a 310 x 110 pile the unfactored ULS lateral resistance is 450 kN per pile at the abutment locations.

The unfactored lateral resistances provided above were developed using the Broms[4] (1964) approach.

The ULS resistances obtained using the above parameter represent unfactored values; in accordance with the Table 6.2 of the CHBDC, a resistance factor of ϕ_{gu} = 0.5 is to be applied in calculating the horizontal resistance. These values provide a limit on the lateral geotechnical resistance when using the p-y curves for design.

Further refinements to the lateral resistance provided above, including generating the p-y curves for the structure, will be required during detailed design.

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and any associated wing walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following preliminary recommendations are made concerning the design of the walls:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular A or Granular B Type II, should be used as backfill behind the walls. Alternatively, Select, free draining 19 mm clear crushed stone granular fill meeting the OPSS.PROV 1004 Type 1 specifications can be used as backfill behind the abutment walls. A Class II nonwoven geotextile meeting the specifications of OPSS 1860 and having a Filtration Opening Size not exceeding 100 microns is to placed over the existing embankment fill and native soil, with overlaps of at least 0.5 m between rolls, prior to placement of the clear stone. If clear stone backfill is used it should only be placed once the wing walls are in place, otherwise some type of restraint (e.g., gabion baskets) would need to be provided perpendicular to the abutments (i.e., at the ends of the excavations) prior to placement of the clear stone. The clear stone backfill should be nominally compacted in 300 mm lifts with vibratory compaction equipment to ensure that all voids have settled out so that no future settlement of the backfill will occur.

- Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).

[4] Broms, Bengt. B., M.ASCE, 1964. Lateral Resistance of Piles in Cohesionless Soils, Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (Case (a) on Figure C6.20 of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the footing (Case (b) on Figure C6.20 of the Commentary to the CHBDC).

6.5.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes, then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the proposed embankment fill and the following unfactored parameters in Table 8 may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Table 8: Static Lateral Earth Pressure Coefficients, Earth Fill or SSM

Soil Type	Internal Angle of Friction (ϕ°)	Soil Unit Weight (γ, kN/m^3)	Coefficients of Earth Pressure		
			Active, Ka	At-Rest, Ko	Passive, Kp
Earth Fill or SSM	30	20	0.33	0.50	3.0

For Case (b), the pressures are based on using engineered granular fill or clear stone and the following unfactored parameters in Table 9 may be used.

Table 9: Static Lateral Earth Pressure Coefficients, Earth Granular A, B Type II and Clear Stone

Soil Type	Internal Angle of Friction (ϕ°)	Soil Unit Weight (γ, kN/m^3)	Coefficients of Earth Pressure		
			Active, Ka	At-Rest, Ko	Passive, Kp
Granular A	35	22	0.27	0.43	3.7
Granular B Type II	35	21	0.27	0.43	3.7
Clear Stone	28	17	0.36	0.53	2.8

Where the wall support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

Where the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the Commentary to the CHBDC.

6.5.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Sections 4.6 and 6.14 of the CHBDC. In this regard, the following guidance should be included in the assessment of lateral earth pressures.

Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.5.1 above, plus the earthquake-induced dynamic earth pressure.

In accordance with Sections 6.14 and C6.14.7.2 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as equal to the site adjusted PGA estimated at the ground surface (i.e., 0.371g for Site Class D for this site; see Section 6.2.3). For structures which allow lateral yielding, k_h is taken as 0.5 times the site adjusted PGA estimated at the ground surface (i.e., 0.19g for Site Class D).

The seismic active pressure coefficients (K_{AE}) provided in Table 10 for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

In accordance with Section C6.14.7.2 of the Commentary to the CHBDC the K_{AE} value for a yielding wall is applicable provided that the wall can move up to 250k_h mm, where k_h is the site-specific PGA as given in Table 10. This corresponds to displacements of about 70 mm for the 2,475-year design earthquake at this site.

Table 10: Seismic Active Pressure Coefficients, K_{AE} for Various Materials

Structure Type	Design Earthquake	Site Specific PGA (g)	Granular A	Granular B Type II	SSM	Clear Stone
Yielding Wall	2,475-year	0.371	0.38	0.38	0.46	0.49
Non-Yielding Wall			0.55	0.55	0.66	0.71

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a\,\gamma\,d + (K_{AE} - K_a)\,\gamma\,(H\text{-}d)\text{, yielding walls}$$

$$\sigma_h(d) = K_o\,\gamma\,d + (K_{AE} - K_a)\,\gamma\,(H\text{-}d)\text{, non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d, (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and

H is the total height of the wall (m).

6.6 Embankment Design and Construction

At the preliminary stage the new alignment to Power Dam Drive has yet to determined. The following preliminary recommendations will need to be refined during detailed design once the preferred alignment has been selected.

6.6.1 Embankment Settlement and Stability

Should new approach embankments be required assuming the ramp embankments are no higher than the existing and are provided with side slopes no steeper than 2H:1V the following should apply:

- The embankments will likely have factors of safety against global instability under both static and seismic loading conditions of at least 1.5 and 1.1, respectively; and,
- The post-paving settlements upon completion of the construction should meet MTO's requirements for non-freeways (e.g., less than 25 mm within 20 m of the bridge abutments). Preloading may be required due the presence of the compressible, but thin, clay layer.

The above guidance is preliminary since it is based on the available existing information (which does not include boreholes advanced through the existing embankments) and must be confirmed during detailed design.

6.6.2 Subgrade Preparation

Any surficial topsoil, organic matter, and softened/loosened soils or fill containing deleterious material should be stripped from within the limits of the footprint of the new embankment, including from the any existing embankment side slopes. All subgrade soils should be proof rolled prior to fill placement.

Any new embankment fill for the approach embankments should be placed and compacted in accordance with OPSS.PROV 206 (Grading) and OPSS.PROV 501 (Compacting). Should the existing embankments be incorporated into the design, benching of the existing embankment side slopes should be carried out to "key in" the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (Benching of Earth Slopes).

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS.PROV 804 (*Temporary Erosion Control*).

6.7 Corrosion and Cement Type

One soil sample was submitted to Eurofins for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The analysis 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 4 of this report, were compared with Table 3 of Canadian Standards Association Standards A23.1-14 (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. The resistivity results in Table 4 of this report, were compared with Table 3.2 of MTO's *Gravity Pipe Design Guideline* (2014) and generally indicate a severe potential for corrosion of exposed ferrous metal at the site, which should be considered in the design.

6.8 Additional Investigation Works

During the future detailed design, further foundation investigation and analysis will be warranted once the proposed realignment of Power Dam Drive has been finalized. It is recommended that the detailed geotechnical investigation include the following:

- An assessment of the thickness and geotechnical properties of the fill, native soils and bedrock along the proposed alignment to supplement the existing borehole information. Odometer testing of the native clay should be carried out to assess embankment construction recommendations. Coring and laboratory testing to assess the quality and strength of the existing bedrock may also be carried out to confirm the geotechnical resistances, depending on the type and condition of rock encountered during the detailed design investigations.
- It may be beneficial, depending on the proposed rehabilitation plan, to carry out Multi-Channel Analysis of Surface Wave (MASW) testing to assess the average shear wave velocity of the 30 m of soil/bedrock beneath at proposed abutment foundation locations. By having the site-specific shear wave velocity profile it may be possible to upgrade the preliminary seismic site class provided.

7.0 CLOSURE

This report was prepared Kenton Power, P.Eng. The report was reviewed by William Cavers, P.Eng. a Senior Geotechnical Engineer with Golder and the Designated MTO Foundations Contact for this project.

Golder Associates Ltd.

Kenton C. Power, P.Eng.
Geotechnical Engineer

William Cavers, P.Eng
Designated MTO Foundations Contact

BW/KCP/WC/hwd

https://golderassociates.sharepoint.com/sites/144785/project files/6 deliverables/3-final/21464403-rev0-fidr final hwy 401-power dam 2022-06-03.docx

Table 11: Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/Risks
Steel H-pile foundations founded on or socketed into rock.	■ Feasible ■ Preferred foundations option for integral abutments.	■ Compatible with integral abutments. ■ High bearing resistance ■ Negligible settlement ■ New piles could provide additional uplift capacity if grouted/cemented into bedrock	■ Requires pre-drilled sockets into the bedrock prior to installing the H-piles for uplift resistance or for integra abutments (depending on underside of pile cap elevation) ■ Driven piles could become hung up in the glacial till and may not reach the bedrock surface	■ Less expensive than caissons.	■ Driven piles may get hung up in the glacial till with low overhead driving energy and therefore provide reduced axial capacity and may require additional piles to be installed ■ Rock socketing could be required which could increase installation costs mobilizing separate equipment and time
Steel pipe pile socketed into rock.	■ Feasible ■ A preferred option from a foundation's perspective	■ May be compatible with integral abutments. ■ High bearing resistance ■ Negligible settlement ■ New piles could provide additional uplift capacity if grouted/cemented into bedrock ■ Drilled in pipe piles including creating the bedrock socket can be accomplished using the down-the-hole hammer installation method	■ Driven piles could become hung up in the glacial till and may not reach the bedrock surface	■ Less expensive than caissons and slightly less expensive than H-piles. ■ Down-the-hole hammer installation may have a higher cost, compared to conventional installation due to number of companies with specialized equipment	■ Driven piles may get hung up in the glacial till and therefore provide reduced axial capacity and may require additional piles to be installed ■ Rock socketing required which could increase installation costs and time
Caissons founded on or socketed into rock	■ Not feasible	■ High bearing resistance ■ Negligible settlement	■ Not compatible for support of the bridge replacement with integral abutments ■ Would require temporary lining for installation ■ Tremie concrete placement required	■ Most expensive option	■ Temporary liners required
New cast-in-place or precast spread footings supported on naïve soil or bedrock	■ Not feasible due to low strength soils and relatively deep excavations below the groundwater to install on bedrock				

MINISTRY OF TRANSPORTATION, ONTARIO

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

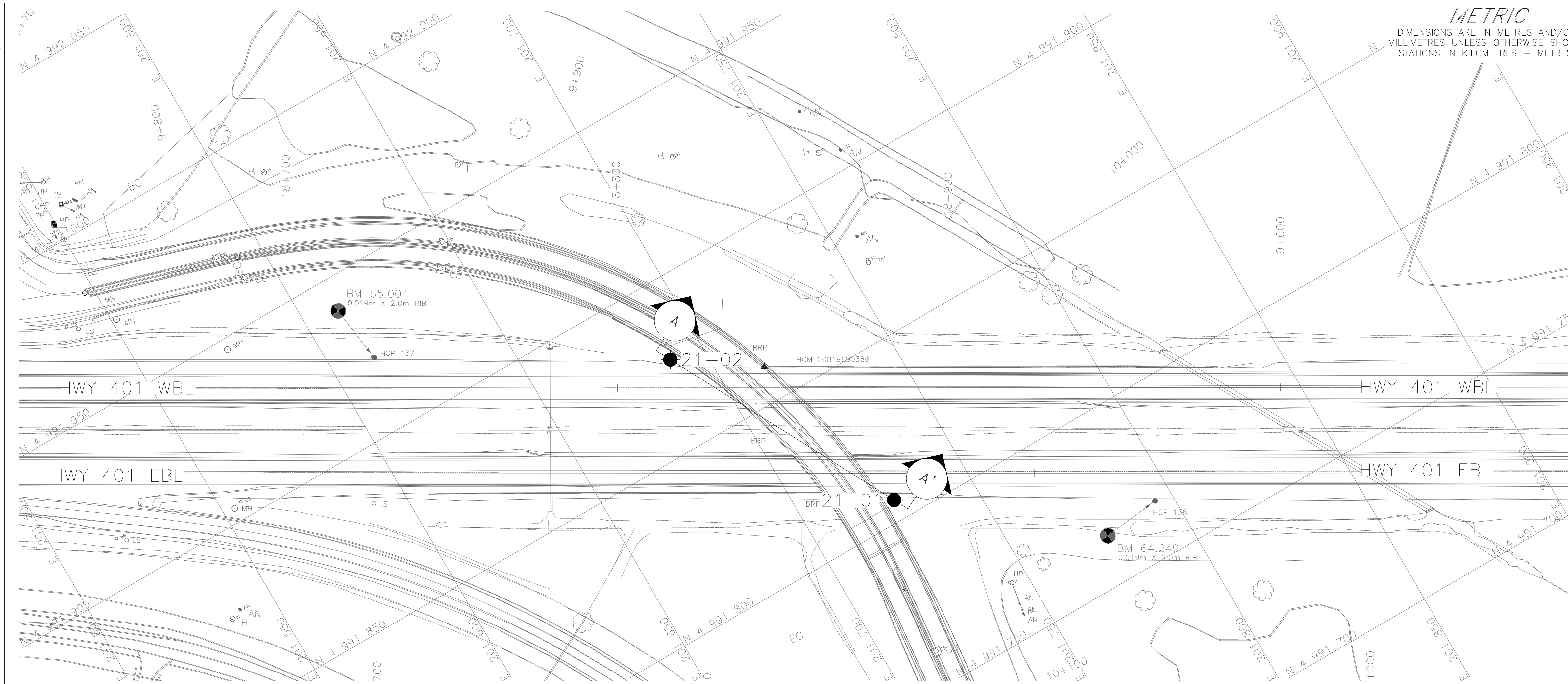
CONT No.
GWP No.

SHEET

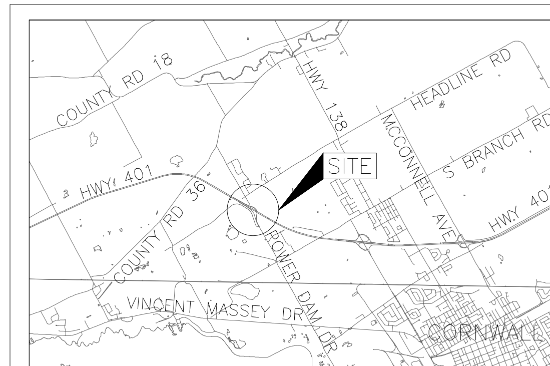
HIGHWAY 401/POWER DAM DRIVE BRIDGE REPLACEMENT

BOREHOLE LOCATIONS AND SOIL STRATIGRAPHY
LAT. 45.058740 LONG. −74.808730

GOLDER
MEMBER OF WSP



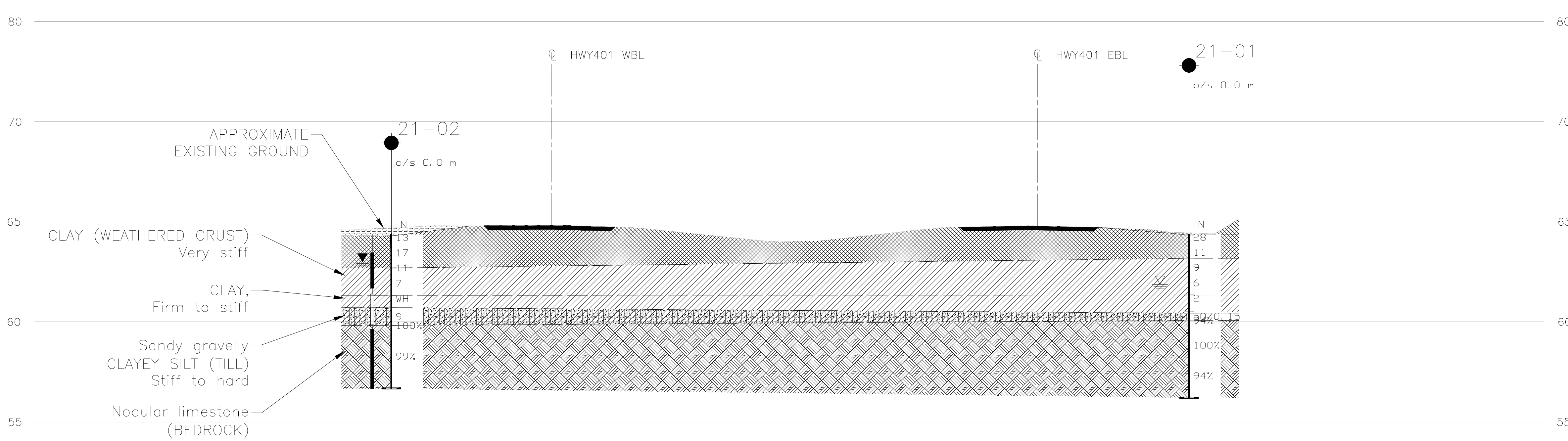
PLAN
SCALE
15 0 15 30 m



KEY PLAN
SCALE
2 0 2 4 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 June 7, 2022
▽	WL upon completion of drilling



PROFILE A−A'
SCALE
5 0 5 10 m

BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 8			
No.	ELEVATION	NORTHING	EASTING
21−01	64.4	4991806.3	201726.8
21−02	64.4	4991876.9	201689.7

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

Base plan provided in digital format by Morrison Hershfield, drawing file no. Basemapping.dwg, received November 10, 2021.

LICENSED PROFESSIONAL ENGINEER W. CAVERS 100077934 09-06-2022 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER K. T. C. POWER 100130652 June 9, 2022 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
A			

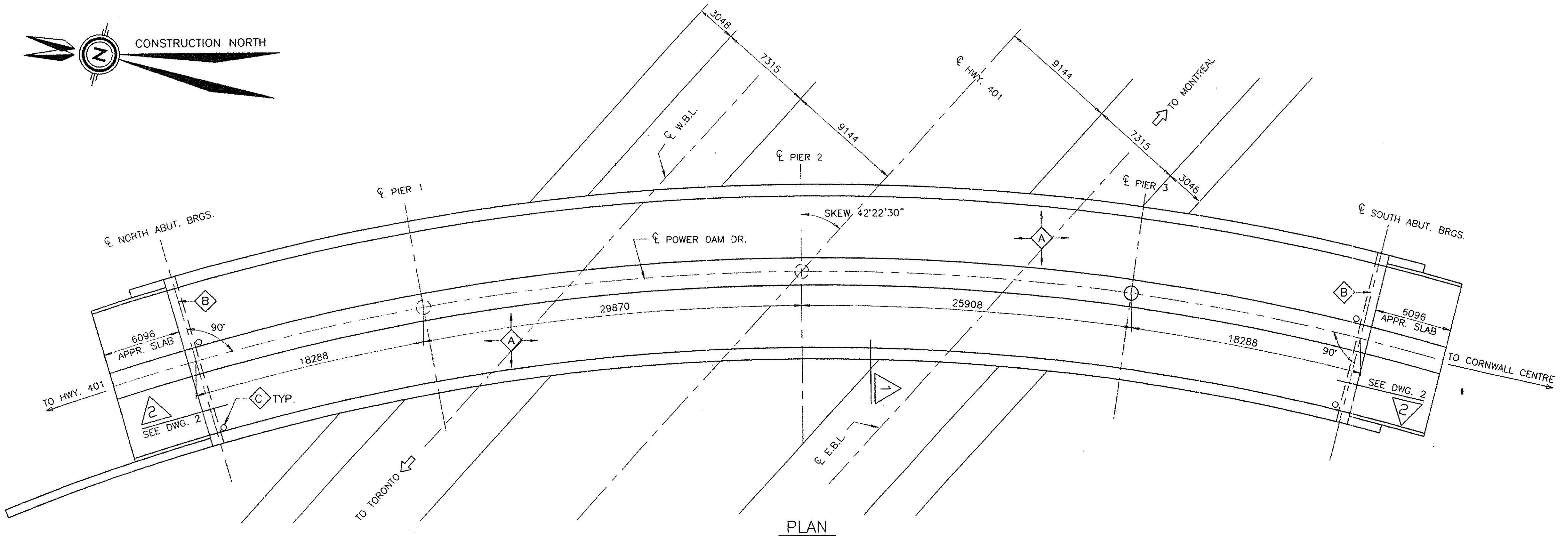
Geocres No. 31G−288

HWY. 401	PROJECT NO. 21464403		DIST. EASTERN
SUBM'D. KCP	CHKD. BW	DATE: 6/13/2022	SITE:
DRAWN: ZS	CHKD. KCP	APPD. WC	DWG. 1

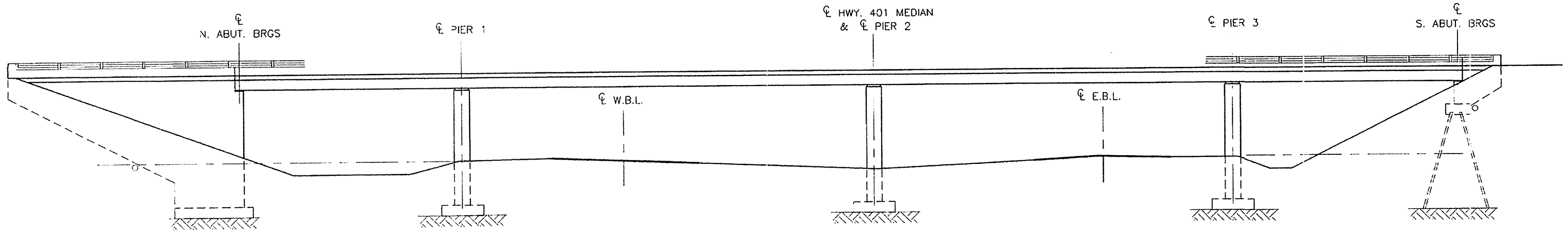
PLOT DATE: June 13, 2022
FILENAME: N:\Active\Spatial_IM\MTO\Hwy401\Mega18_PowerDamDr\99_PROJ\21464403_21464403_BridgeReplacement\40_PROD\0001_Preliminary Foundations Investigation\21464403-0001-BG-0001.dwg

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

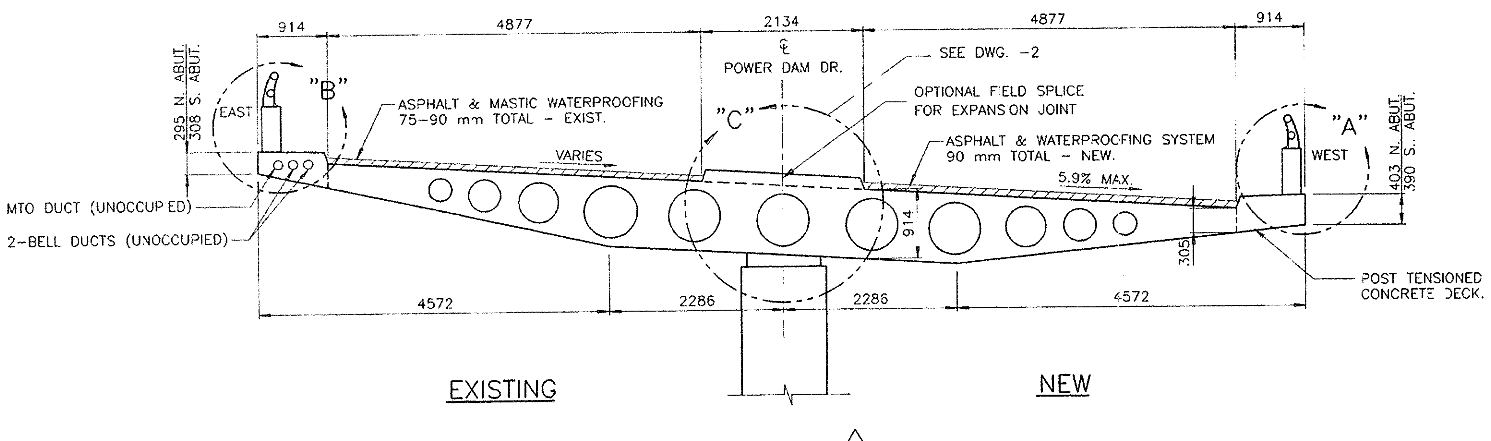
DIST 42 OTTAWA CONT No . 97-72 WP No 80-91-03		
POWER DAM DR. U'PASS (HWY. 401) GENERAL ARRANGEMENT	SHEET	5



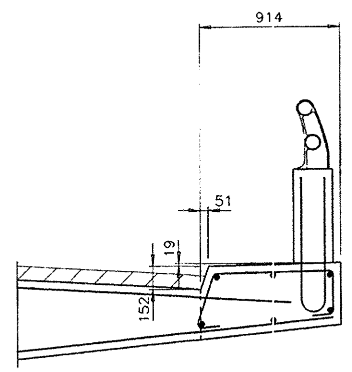
PLAN
SCALE 1:200



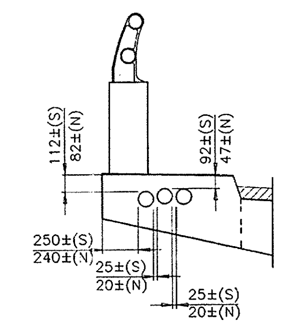
ELEVATION
SCALE 1:200



SECTION 1
SCALE 1:50



DETAIL "A"
SCALE 1:25



DETAIL "B"
SCALE 1:25

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

GENERAL NOTES :

CLASS OF CONCRETE

CLASS OF CONCRETE.......... 30 MPa

CLEAR COVER TO REINFORCING STEEL

CLEAR COVER TO REINFORCING STEEL SHALL BE 70 ± 20 mm UNLESS OTHERWISE NOTED.

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH PREFIX 'C' DENOTES COATED BARS.

CONSTRUCTION NOTES

- SAWCUTS IN CONCRETE, WHEREVER DESIGNATED, SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS.
- THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
- THE CONTRACTOR SHALL CHECK ALL RELEVANT DIMENSIONS AND ELEVATIONS OF EXISTING WORK PRIOR TO FABRICATION OF THE JOINT ASSEMBLIES. DIMENSIONS AND ELEVATIONS SHALL BE ADJUSTED AS REQUIRED TO SUIT THE PROPOSED WORK.
- C.J. DENOTES CONSTRUCTION JOINT.

WORK DESCRIPTION :

A — REMOVE ASPHALT PAVEMENT AND WATERPROOFING FROM DECK. REMOVE DETERIORATED AND DELAMINATED CONCRETE FROM DECK, IF REQUESTED BY THE CONTRACT ADMINISTRATOR (EXTRA WORK). WATERPROOF AND REPAVE.

B — MODIFY EXPANSION JOINTS AS DETAILED.

C — INSTALL DRAINAGE TUBES IN DECK.

LIST OF DRAWINGS

31-180/R2-1	GENERAL ARRANGEMENT.
31-180/R2-2	EXPANSION JOINT DETAILS AND STAGING.
31-180/R2-3	JOINT ANCHORAGE AND ARMOURING. WITH INJECTION HOSE SYSTEM-ASSEMBLY.
31-180/R2-4	JOINT ANCHORAGE AND ARMOURING. WITH INJECTION HOSE SYSTEM-DETAILS.

REFERENCE DRAWINGS

31-180-1-B TO 31-180-13-B
CONT. 81-302

REGISTERED PROFESSIONAL ENGINEER
Q. M. ISLAM
PROVINCE OF ONTARIO

APPLICABLE STANDARD DRAWINGS

O.P.S.D.	3906.02	BRIDGE DECK WATERPROOFING
O.P.S.D.	3951.00	DRAINAGE ON ASPHALT WEARING SURFACES ON EXISTING DECKS

REVISIONS

DESCRIPTION

DESIGN SNC	CHK Q.I	CODE OHBDC'91	LOAD OHBD	DATE JAN/98	
DRAWN M.M.	CHK TMN	SITE 31-180	STRUCT	SCHEME R2	DWG 1

AutoCAD File Name : H:\proj-cms\s031-180\cad\31-180r21 Date : 02/03/98 14:42:40

APPENDIX A

Lists of Abbreviations and Symbols

Record of Boreholes 20-01 to 20-02

Bedrock Core Photographs, Figures A1 to A4

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 Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
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)
≤ 10	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 cm^2 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, Gs)
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)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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 grainsize. 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.

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
υ	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 ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

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

(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
$C_{\alpha(e)}$	secondary compression index
C_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 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

GOLDER
MEMBER OF WSP

RECORD OF BOREHOLE No 21-01

SHEET 1 OF 2

METRIC

PROJECT 21464403-1000

G.W.P. 4092-19-00

LOCATION N 4991806.3; E 201726.8 MTM NAD 83 ZONE 8 (LAT. 45.058740; LONG. -74.808730)

ORIGINATED BY RI

DIST Eastern HWY 401

BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core

COMPILED BY ZS

DATUM Geodetic

DATE October 27, 2021

CHECKED BY KCP

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
64.4	GROUND SURFACE					
0.0	(SM) Gravelly silty sand, contains asphalt and organic matter (FILL) Compact Brown to grey-brown Moist	1	SS	28		
		2	SS	11		25 46 (29)
63.2 / 1.2	(CH) CLAY, highly fissured, thin laminations of silty sand (WEATHERED CRUST) Very stiff Grey, mottled brown w>PL	3	SS	9		0 1 31 68
		4	SS	6	▽	
61.4 / 3.1	(CH) CLAY Stiff Grey w>PL	5	SS	2		
60.5 / 3.9	(CL) Sandy gravelly CLAYEY SILT (TILL) Grey w=PL	6	SS	50/0.15		21 27 33 19
60.1 / 4.3	Nodular limestone (BEDROCK) Bedrock cored from depths 4.3 m to 8.2 m For bedrock coring details refer to Record of Drillhole 21-01	1	RC	REC 100%		RQD = 94%
		2	RC	REC 100%		RQD = 100%
		3	RC	REC 95%		RQD = 94%
56.2 / 8.2	END OF BOREHOLE NOTES: 1. Water level measured at a depth of 2.5 m (Elev. 61.9 m) prior to commencing of bedrock coring					

ELEVATION SCALE: 64, 63, 62, 61, 60, 59, 58, 57

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75

UNIT WEIGHT γ kN/m³

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

RECORD OF DRILLHOLE: 21-01

PROJECT: 21464403-1000
SHEET 2 OF 2
LOCATION: N 4991806.3 ;E 201726.8
DRILLING DATE: October 27, 2021
DATUM: Geodetic
INCLINATION: -90° AZIMUTH: ---
DRILL RIG: CME 55
DRILLING CONTRACTOR: Downing Drilling

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA (DIP w.r.t. CORE AXIS; TYPE AND SURFACE DESCRIPTION; Jr; Ja)	HYDRAULIC CONDUCTIVITY K, cm/sec	WEATHERING INDEX	FEATURES	
		BEDROCK SURFACE		60.08										
5	Rotary Drill NQ Core	Nodular limestone (BEDROCK), with interlaminations of black shale Fresh Thinly to medium bedded medium to dark grey Fine to medium grained Non-porous Excellent quality Very strong - Broken core from 4.3 m to 4.4 m		4.32	1	100								UCS = 127 MPa
6					2	100								
7–8		- Broken core/lost cose from 7.3 m to 7.4 m		56.22	3	100								
		END OF DRILLHOLE		8.18										

Depth scale: 9, 10, 11, 12, 13, 14

DEPTH SCALE
1 : 50

GOLDER
MEMBER OF WSP

LOGGED: RI
CHECKED: KCP

GOLDER MEMBER OF WSP

RECORD OF BOREHOLE No 21-02

SHEET 1 OF 2 **METRIC**

PROJECT 21464403-1000

G.W.P. 4092-19-00 LOCATION N 4991876.9; E 201689.7 MTM NAD 83 ZONE 8 (LAT. 45.059370; LONG. -74.809210) ORIGINATED BY RI

DIST Eastern HWY 401 BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core COMPILED BY ZS

DATUM Geodetic DATE October 27-28, 2021 CHECKED BY KCP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
64.4	GROUND SURFACE						
0.0 / 0.1	(SM) gravelly silty sand, contains organic matter (TOPSOIL) Dark brown Moist	1	SS	13	64		
	(SM) gravelly silty sand, trace to some clay, contains thin beds of silty clay and organic matter (rootlets) (FILL) Compact Brown to dark brown Moist	2	SS	17	63		
62.7 / 1.7	(CH) CLAY, highly fissured (WEATHERED CRUST) Very stiff Grey-brown w>PL	3	SS	11			
		4	SS	7	62		
61.4 / 3.1	(CH) CLAY, contains thin laminations of silty sand firm Grey-brown to grey w>PL	5	SS	WH	61	w_P–w_L plotted	0 1 23 76
60.7 / 3.7	(CL) Sandy gravelly CLAYEY SILT (TILL) Stiff to hard Grey w=PL	6	SS	9	60		
59.9 / 4.6	Nodular limestone (BEDROCK) Bedrock cored from depths 4.6 m to 7.7 m For bedrock coring details refer to Record of Drillhole 21-01	1	RC	REC 100%	59		RQD = 100%
		2	RC	REC 100%	58 / 57		RQD = 99%
56.7 / 7.7	END OF BOREHOLE NOTES: 1. Water level measured in screen at a depth of 1.1 m (Elev. 63.3 m) on November 4, 2021. 2. Water level measured in screen at a depth of 1.4 m (Elev. 63.0 m) on June 7, 2022.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa 20 40 60 80 100 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³.

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

PROJECT: 21464403-1000

RECORD OF DRILLHOLE: 21-02

SHEET 2 OF 2

LOCATION: N 4991876.9 ;E 201689.7

DRILLING DATE: October 27-28, 2021

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: Downing Drilling

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH RETURN	FEATURES
		BEDROCK SURFACE	59.83			
5, 6, 7	Rotary Drill, NQ Core	Nodular limestone (BEDROCK), with interlaminations of black shale Fresh Thinly to medium bedded Medium to dark grey Fine to medium grained Non-porous Excellent quality Very strong	4.55	1	100	Silica Sand UCS = 109 MPa
				2	100	Bentontie
			56.68 7.70			
8		END OF DRILLHOLE				

Depth scale: 9, 10, 11, 12, 13, 14

Column headings: RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20), R.Q.D. % (80 60 40 20), FRACT. INDEX PER (5 10 15 20), DISCONTINUITY DATA (DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja), HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}), WEATHERING INDEX (W1 W2 W3 W4 W5 W6), SYMBOLIC LOG

DEPTH SCALE
1 : 50

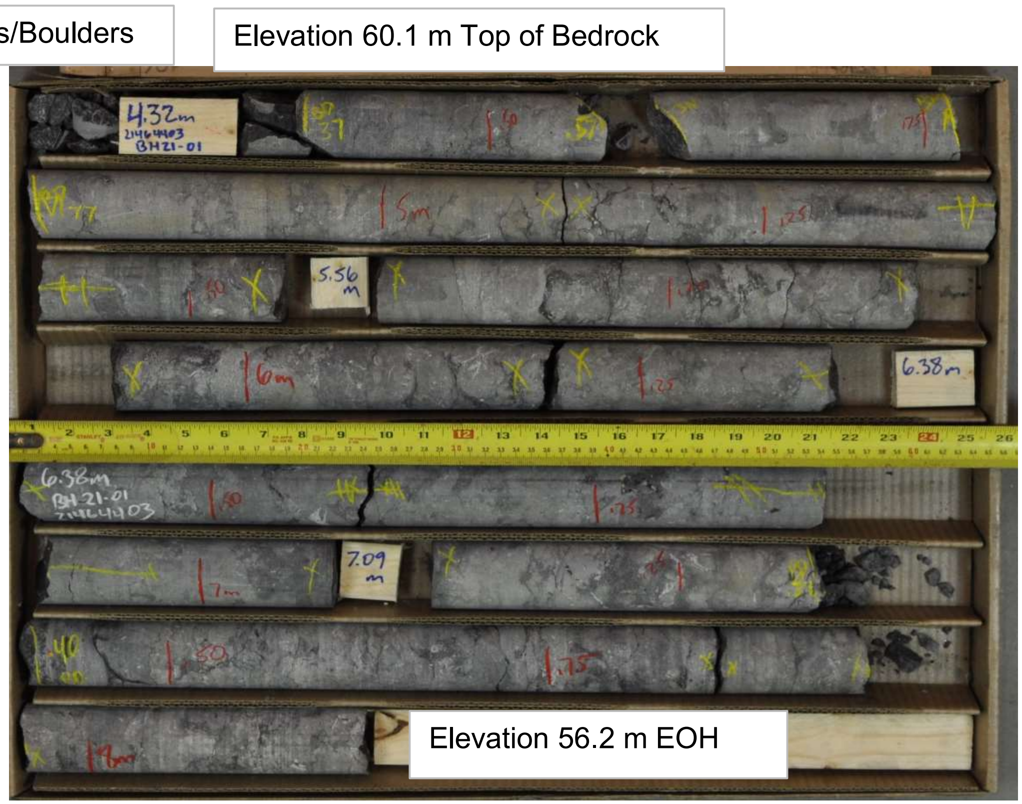
GOLDER
MEMBER OF WSP

LOGGED: RI
CHECKED: KCP

BH 21-01 (Dry)

Cored Length of 60.1 to 56.2 m

Core Box 1 to 2 of 2



Foundation Investigation

Replacmenet of Highway 401 / Power Dam Drive Underpass

GWP: 4092-19-00

Ottawa, ON

GOLDER
MEMBER OF WSP

Project No.	21464403-1000
Drawn:	BW
Date:	2021-10-30
Checked:	KP
Review:	WC

Figure A1

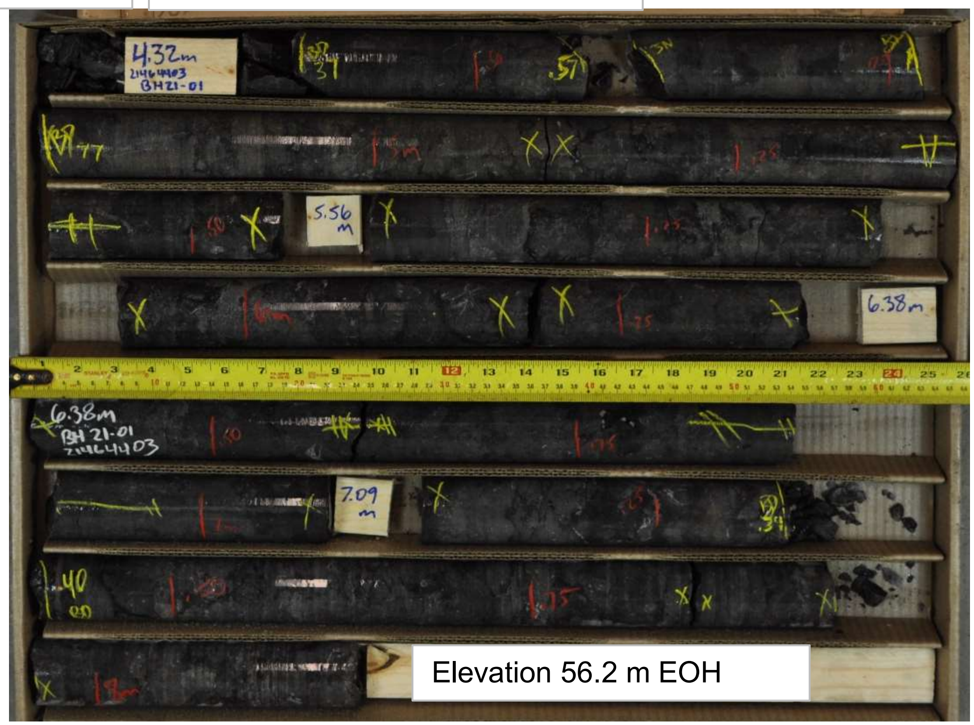
BH 21-01 (Wet)

Cored Length of 60.1 to 56.2 m

Core Box 1 to 2 of 2

Cobbles/Boulders

Elevation 60.1 m Top of Bedrock



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Foundation Investigation

Replacmenet of Highway 401 / Power Dam Drive Underpass

GWP: 4092-19-00

Ottawa, ON

Project No.	21464403-1000
Drawn:	BW
Date:	2021-10-30
Checked:	KP
Review:	WC

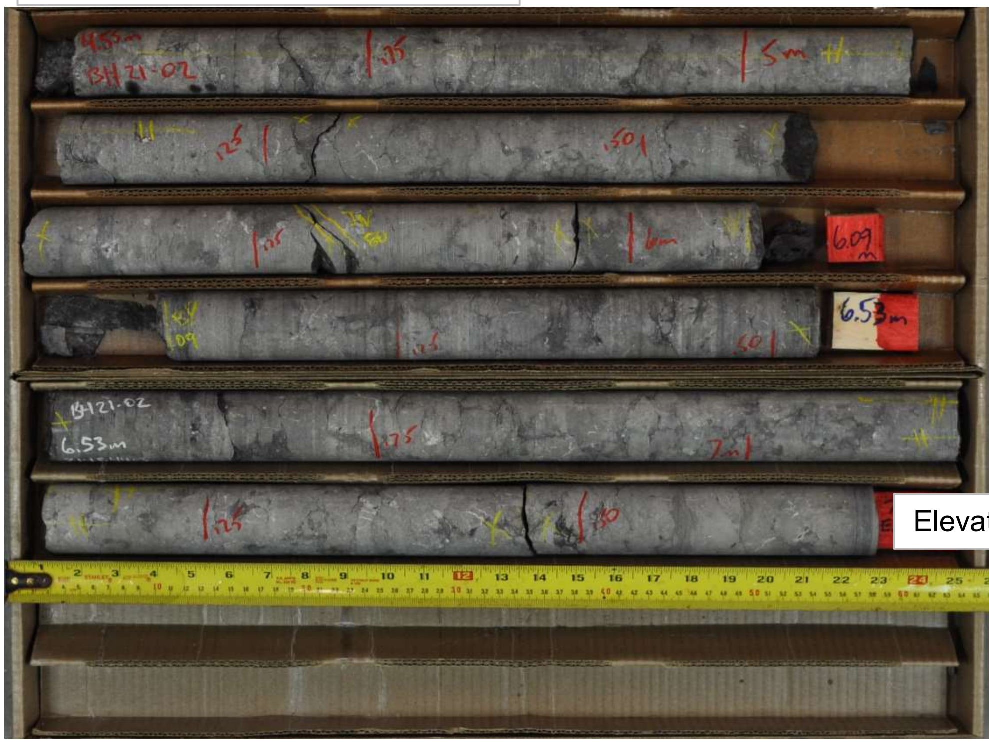
Figure A2

BH 21-02 (Dry)

Cored Length of 59.9 to 56.7 m

Core Box 1 to 2 of 2

Elevation 59.9 m Top of Bedrock



Elevation 56.7 m EOH

GOLDER MEMBER OF WSP	**Foundation Investigation** **Replacmenet of Highway 401 / Power Dam Drive Underpass** **GWP: 4092-19-00** **Ottawa, ON**	Project No. 21464403-1000 Drawn: BW Date: 2021-10-30 Checked: KP Review: WC	**Figure A3**

BH 21-02 (Wet)

Cored Length of 59.9 to 56.7 m

Core Box 1 to 2 of 2

Elevation 59.9 m Top of Bedrock

Elevation 56.7 m EOH

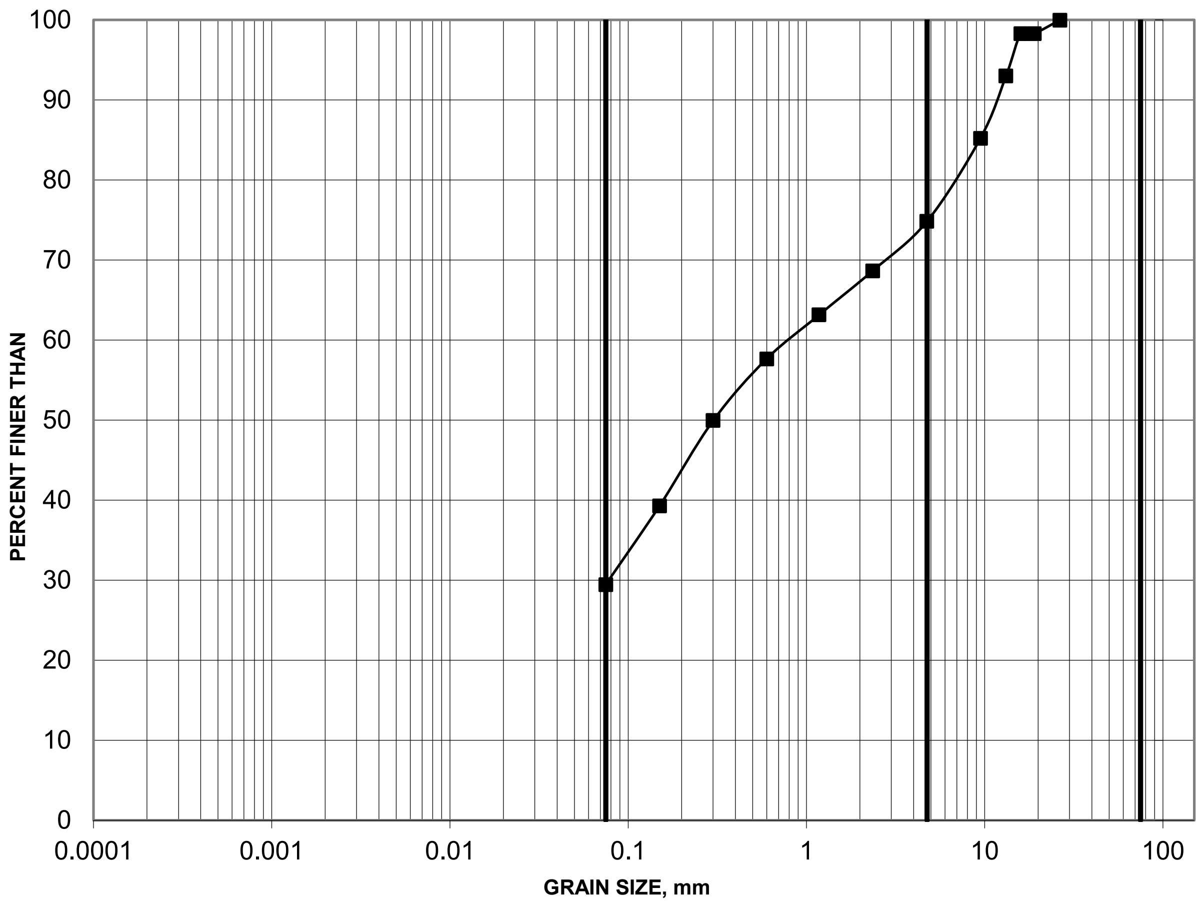
GOLDER MEMBER OF WSP	**Foundation Investigation** **Replacmenet of Highway 401 / Power Dam Drive Underpass** **GWP: 4092-19-00** **Ottawa, ON**	Project No.: 21464403-1000 Drawn: BW Date: 2021-10-30 Checked: KP Review: WC	**Figure A4**

APPENDIX B

Laboratory Test Results
Figures B1 to B4

GRAIN SIZE DISTRIBUTION	FIGURE B1

GRAVELLY SILTY SAND (FILL)



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
21-1	2A	0.76-1.37	25	46	29	

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Project: 21464403/1000

Created by:	BW
Checked by:	MI

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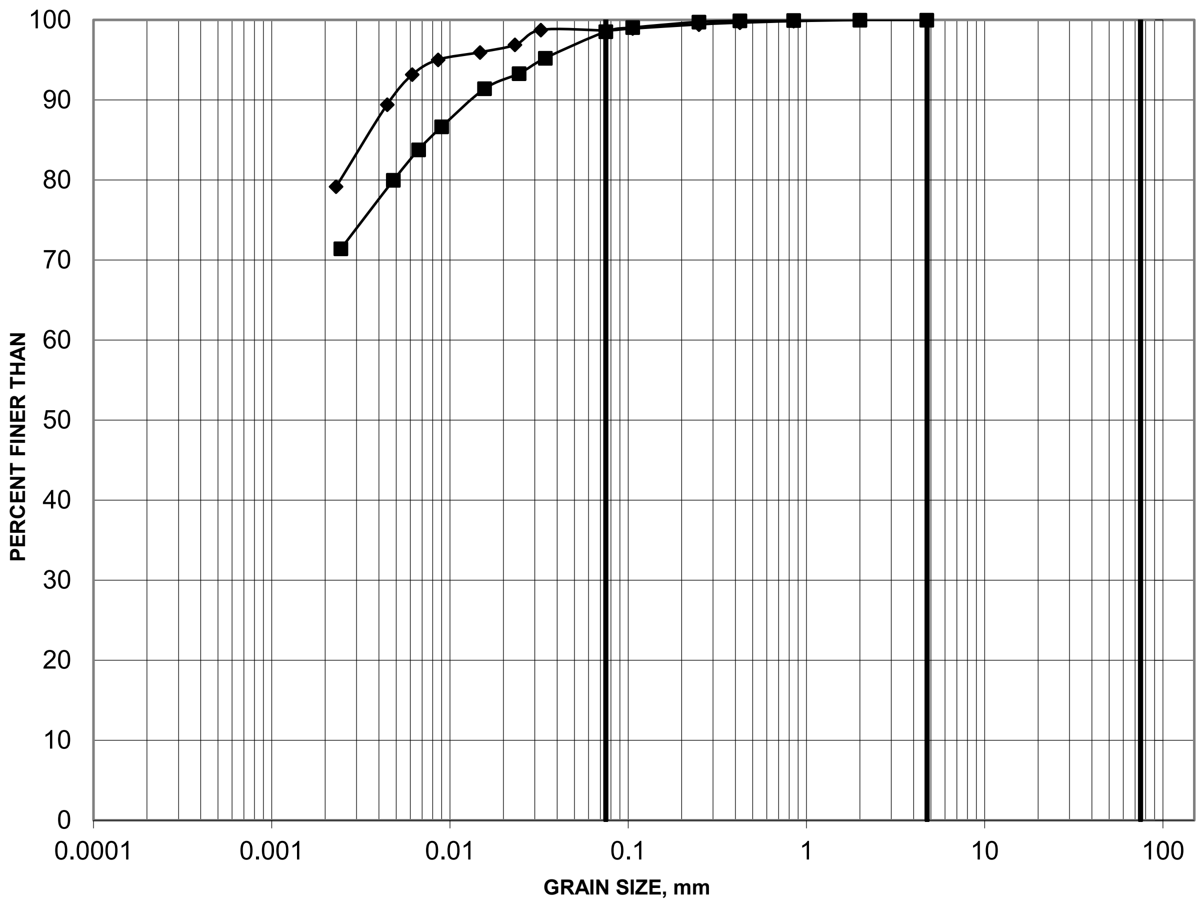
GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAY



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

				Constituents (%)			
	Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
■	21-1	3	1.52-2.13	0	1	31	68
◆	21-2	5	3.05-3.66	0	1	23	76

GOLDER
MEMBER OF WSP

Project: 21464403/1000

Created by: BW
Checked by: MI

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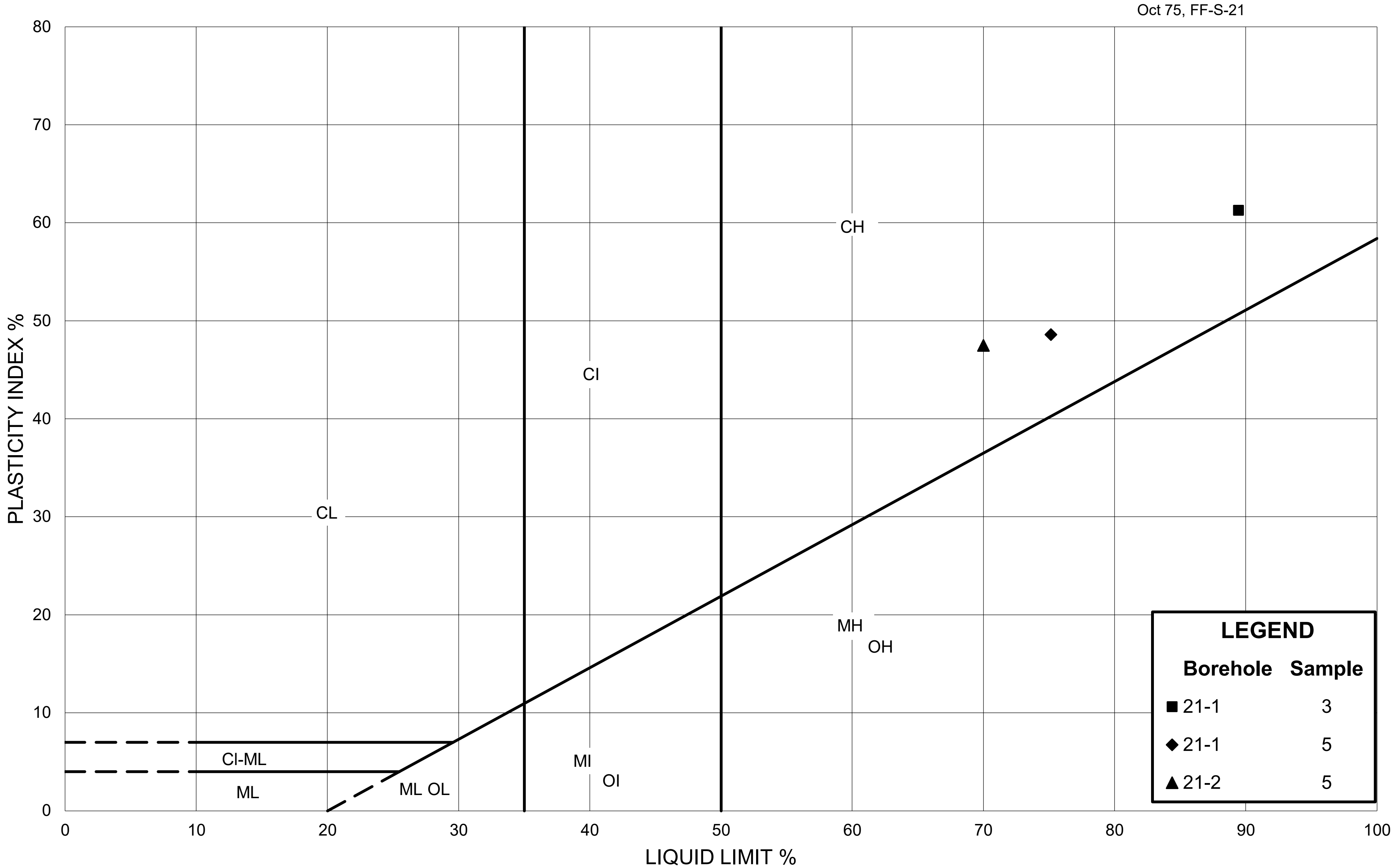


Ministry of Transportation

Ontario

PLASTICITY CHART

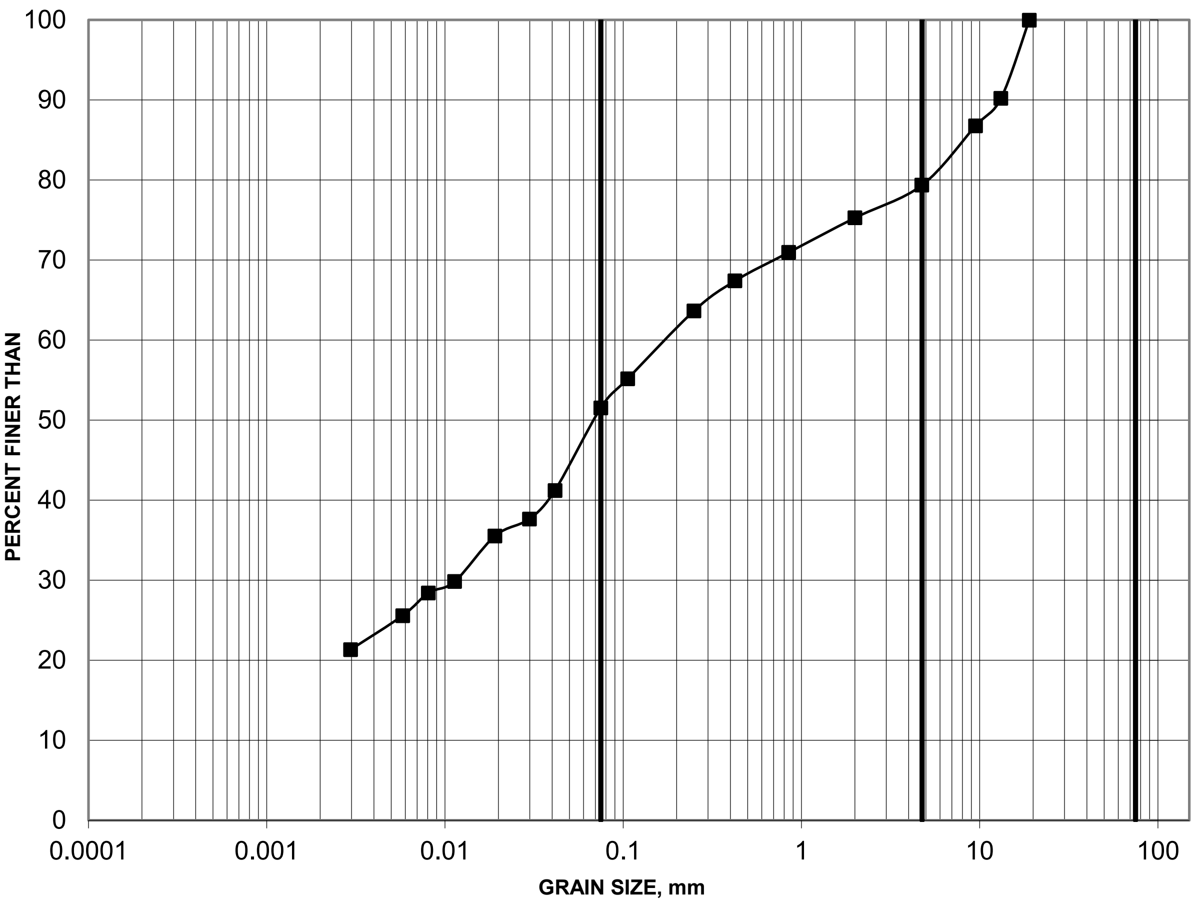
CLAY

Figure:	B3
Project:	21464403/1000
Created By: BW	Checked By: MI

GRAIN SIZE DISTRIBUTION

FIGURE B4

GLACIAL TILL



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

			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
21-1	6	3.96-4.27	21	27	33	19

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Project: 21464403/1000

Created by: BW
Checked by: MI

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APPENDIX C

Results of Chemical Analysis

Eurofins Environment Testing Report No. 1966621

Certificate of Analysis

Client:	Golder Associates Ltd (Ottawa) 1931 Robertson Road, Ottawa, Ontario xxx	Report Number:	1966621
Attention:	Mr. Kenton Power	Date Submitted:	2021-11-10
PO#:		Date Reported:	2021-11-17
Invoice to:	Golder Associates Ltd	Project:	21464403
		COC #:	882809

Group	Analyte	MRL	Units	Guideline	1594951
				Lab I.D.	1594951
				Sample Matrix	Soil
				Sample Type	
				Sampling Date	2021-10-27
				Sample I.D.	21-2 sa3
Anions	Cl	0.002	%		<0.002
	SO4	0.01	%		0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		1.36
	pH	2.00			7.80
	Resistivity	1	ohm-cm		741

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: Borehole 21-02 looking east along the westbound ditchline

Photograph 2: Borehole 21-02 looking west along the Highway 401 westbound ditchline

Photograph 3: Near Borehole 21-02 looking south across Highway 401 towards Borehole 21-01

Photograph 4: Borehole 21-01 looking west along eastbound Highway 401 ditchline

Photograph 5: Borehole 21-01 looking east along eastbound the Highway 401 ditchline

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