UPDATED REPORT

Geotechnical Investigation
Proposed Hydro One Operations Facility

3440 Frank Kenny Road, Ottawa, Ontario

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed Hydro One operations facility to be located at 3440 Frank Kenny Road in Ottawa, Ontario.

This report was previously issued under report number 11-1122-0129-2000 in January 2012 by Golder Associates Ltd. Information contained within this report was prepared prior to, and during the transition period of the acquisition of WSP. This report provides updated geotechnical guidance for Phase 2 of the proposed facility and supersedes the previously issued report. Further, this report is based solely on the results of the previous geotechnical investigations, with the exception of updated water levels, and the site conditions may have changed due to construction or other activities on the site since those investigations were completed.

The purpose of the geotechnical investigation was to assess the subsurface conditions at the site by means of a limited number of test pits and boreholes.

Based on an interpretation of the factual information available for this site, a general description of the subsurface conditions across the site is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Plans are being prepared for the construction of a Hydro One operations facility to be located at 3440 Frank Kenny Road in Ottawa, Ontario (see Key Plan, Figure 1).

The following is known about the existing property:

- The overall site measures approximately 145 metres by 360 metres in plan area.
- The northern part of the site (3406 Frank Kenny Road) is occupied by M. L. Bradley Bus Lines (Bradley) and contains several buildings.
- The southern part of the site (3440 Frank Kenny Road) is occupied by a residential dwelling and is agricultural land.
- The overall site topography is relatively flat.

It is understood that the proposed operations facility is to be constructed in two phases. The first phase will include:

- A temporary office building located on the western portion of the 3440 Frank Kenny Road property. The temporary office building will measure about 15 metres by 20 metres in plan area, will be one storey in height, and will be of slab-on-grade construction (i.e., no basement level).
- A general storage building to be located on the north side of the 3440 Frank Kenny Road property. The general storage building will measure about 14 metres by 22 metres in plan area, will be one storey in height, and will be of slab-on-grade construction (i.e., no basement level).
- Gravel surfaced roadways and parking areas.

It is also understood that the grades will not be raised within the phase 1 area.

The second phase will include:

- A permanent office/storage building located on the south side of the 3440 Frank Kenny Road property. The office building will measure about 32 metres by 66 metres in plan area (including the covered vehicle storage area), will be one storey in height, and will be of slab-on-grade construction (i.e., no basement level).
- The office/storage building will be provided with a storage ramp on the north side.
- A concrete pad for placement of two fire water storage tanks.
- A storm water management pond in the southern corner of the site.
- Asphalt and gravel surfaced roadways and parking areas. The asphalt parking area includes lanes for heavy vehicle (truck) traffic.

It is also understood that the grades will be raised by up to about 1.5 metres within the phase 2 area.

Published geological mapping indicates that the subsurface conditions at the site consist of silty clay. The bedrock surface is expected to be at a depth of 5 to 10 metres below ground surface at the northern portion of the site and 3 to 5 metres below ground surface at the southern portion of the site.

Geological bedrock mapping indicates that the site is located near the contact between two bedrock formations. At the northern portion of the site, the bedrock is indicated to consist of interbedded limestone and shale of the Lindsay Formation while, at the southern portion of the site, the bedrock is indicated to consist of shale of the Billings Formation.

3.0 PROCEDURE

The field work for this investigation was carried out between October 31 and November 1, 2011. During this period, a total of seven boreholes (numbered BH 11-1 to BH 11-7) and five test pits (numbered TP 11-1 to TP 11-4) were put down at the approximate locations shown on Figure 2.

The boreholes were advanced using a track-mounted hollow-stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The boreholes were advanced to depths which vary from 2.0 to 7.0 metres below existing ground surface.

Within the boreholes, standard penetration tests were carried out at regular intervals of depth and samples of the soils encountered were recovered using drive open sampling equipment. In situ vane testing was carried out where possible in the silty clay to determine the undrained shear strength of this soil unit. In addition, two relatively undisturbed, 73-millimetre diameter thin-walled Shelby tube samples of the silty clay were obtained using a fixed piston sampler.

Standpipes were sealed into boreholes 11-3 and 11-5 to allow subsequent measurement of the stabilized groundwater level at the site.

The test pits were excavated using a rubber-tired backhoe supplied and operated by Glenn Wright Excavating of Ottawa, Ontario. The test pits were advanced to depths ranging from approximately 1.6 to 2.4 metres below the existing ground surface.

The soils exposed on the sides of the test pits were classified by visual and tactile examination. The groundwater seepage conditions were observed in the open test pits and the test pits were loosely backfilled upon completion of excavating and sampling.

The subsurface conditions encountered in the test pits are shown on Table 1 - Record of Test Pits.

The field work was supervised by an experienced technician from our staff who located the boreholes and test pits, directed the drilling and excavating operations, logged the boreholes and test pits, took custody of the samples, and carried out the in situ testing. The soil samples obtained during the field work were brought to our laboratory for further examination by the project engineer and for laboratory testing. Geotechnical laboratory testing included determination of water content (D-2216; LS-701) and Atterberg limits (D-4318; LS703/704).

One sample of soil from borehole 11-5 was submitted to Exova Accutest Laboratories Ltd. for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The boreholes and test pits were selected, staked in the field, and subsequently surveyed by Golder Associates personnel. The positions and ground surface elevations at the borehole and test locations were determined using a Trimble R8 GPS survey unit. The elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes during the current investigation are shown on the Record of Borehole Sheets in Appendix A. The subsurface conditions encountered in the test pits are shown on Table 1 – Record of Test Pits. The results of the laboratory water content and Atterberg limit testing on the selected soil samples are given on the Record of Borehole Sheets. The results of the basic chemical analyses are provided in Appendix B.

In general, the subsurface conditions at this site consist of surficial topsoil or fill (where present) overlying sensitive silty clay and glacial till, with the underlying shale bedrock surface varying from about 3 to 4 metres depth at the south portion of the site and greater than 7 metres depth at the north portion of the site.

The following sections present a more detailed overview of the subsurface conditions encountered in the boreholes and test pits advanced during the present investigation. The subsurface conditions encountered in the monitoring well (MW 11-1) are provided on the Record of Borehole Sheet in Appendix A, but are not discussed in the following sections.

GHD carried out a separate hydrogeological investigation which included the installation of four monitoring wells in the Phase 2 area. The results of this investigation are contained in the following reports:

Hydrogeological Assessment, Proposed Development, Orleans Station Yard, 3440 Frank Kenny Road, Navan Ontario, GHD Report Number 12575389(1), dated June 24, 2022; and,

Hydrogeological Assessment - Amendment, Groundwater Level Monitoring, Proposed Development, Orleans Operations Centre (OC), 3440 Frank Kenny Road, Navan Ontario, GHD Reference Number 12575389-Let-3-Spence, dated August 5, 2022.

The GHD monitoring well locations are shown on Figure1 and the stratigraphic and instrumentation logs are provided in Appendix C. The water level information from the GHD records is included below in Section 4.6.

4.2 Topsoil and Fill

A surficial topsoil layer exists at all of the test pit and borehole locations, with the exception of boreholes 11-1 and 11-4. The topsoil varies from about 80 to 150 millimetres in thickness.

Fill materials exist at the ground surface in boreholes 11-1 and 11-4. At these locations, the fill materials are about 310 and 150 millimetres in thickness, respectively. The fill materials consist of clayey topsoil, sand, organic matter, and crushed stone.

4.3 Silty Clay

The topsoil and fill materials are underlain by a deposit of sensitive silty clay. The upper portion of the deposit has been weathered to a stiff grey brown crust. Towards the south (i.e., Phase 2), the entire deposit has been weathered and extends to about 2.0 to 2.7 metres below the existing ground surface. Towards the north (i.e., Phase 1), the weathered zone extends to about 2.7 to 3.1 metres below the existing ground surface.

The results of in-situ vane testing carried out in the lower portions of the weathered crust gave undrained shear strengths ranging from 44 to 69 kilopascals. Standard penetration tests carried out within the weathered crust gave ‘N’ values ranging from 1 to 12 blows per 305 millimetres of penetration. The results of this in situ testing indicate a firm to very stiff (but generally stiff) consistency. The measured water content of the weathered crust ranges from approximately 30 to 82 percent.

In boreholes 11-1 and 11-3 (i.e., Phase 1), the silty clay below the depth of weathering is grey in colour (borehole 11-2 did not fully penetrate the weathered crust). The unweathered silty clay was fully penetrated in borehole 11-3 and was about 1.2 metres in thickness (i.e., extending down to a depth of about 4.1 metres). The unweathered silty clay was not fully penetrated in borehole 11-1 but was proven to a depth of about 7.0 metres.

The results of in-situ vane testing in the silty clay gave undrained shear strengths as shown below.

Borehole No.	Depth (m)	Su (kPa)	Remoulded Strength (kPa)	Sensitivity
11-1	4	30	5	6.0
	4.3	35	6	5.8
	5.5	28	2	14.0
	5.8	45	6	7.5
11-3	3.8	25	4	6.3
11-4	1.7	44	10	4.4
	2	68	11	6.2
	2.3	70	15	4.7

The results of Atterberg limit testing carried out on two samples of the grey silty clay gave plasticity index values of 58 and 63 percent and liquid limit values of 89 and 93 percent, indicating high plasticity soil. The measured water content of the two grey silty clay samples were 83 and 88 percent, which are slightly below the measured liquid limits. The measured plasticity and water content of the silty clay are generally consistent with similar soil deposits in the area.

4.4 Glacial Till

Glacial till was encountered underlying the silty clay (where fully penetrated) in all borehole locations. The glacial till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand and shale fragments. The glacial till was fully penetrated in four of the boreholes and varied in thickness from about 0.3 to 1.7 metres. In borehole 11-3, the glacial till was not fully penetrated, but was proven to a depth of about 5.6 metres prior to the borehole being terminated.

Standard penetration test 'N' values for this material ranged from 11 to 38 blows per 305 millimetres of penetration, which indicates a compact to dense state of packing for this deposit. However, the higher 'N' values likely reflect the presence of cobbles and boulders, rather than the actual state of packing of the soil matrix.

4.5 Bedrock

Bedrock was encountered underlying the glacial till on the south of the site (i.e., Phase 2) in boreholes 11-4 to 11-7 (inclusive). The depth to bedrock ranges from about 3.1 to 4.3 metres below the existing ground surface.

In these boreholes, the upper portion of the bedrock is highly weathered and the boreholes were advanced into the bedrock by up to an additional 0.5 to 2.4 metres prior to the boreholes being terminated.

The bedrock consists of black shale. Published geological mapping indicates that this shale bedrock is of the Billings Formation.

4.6 Groundwater

The groundwater levels (GWL's) recorded in the piezometers and monitoring wells installed at the site are summarized in the following table:

Hole Designation	Approximate Screen Depth Interval (m)	Screen Strata	Date	GWL Depth below Ground Surface (m)	GWL Elev (m)*
11-3	4.2 – 4.9	Glacial Till	Nov. 14, 2011	1.1	84.4
11-5	3.4 – 4.0	Glacial Till/Bedrock	Nov. 1, 2011	1.3	84.4
DBW001	1.5 – 4.0	Clay/Clayey Gravel	Apr. 19, 2022	1.0	85.6
DBW002	0.9 – 3.1	Clay/Gravelly Clay	Apr. 19, 2022	0.1	85.5
DBW003	1.5 – 3.1	Clay/Clayey Gravel	Apr. 19, 2022	0.2	85.4
DBW004	1.2 – 3.7	Clay	Apr. 19, 2022	0.4	84.7

* The water levels shown for the GHD monitoring wells are the maximum recorded during the monitoring period from April 19, 2022 to July 7, 2022.

Groundwater levels are expected to fluctuate seasonally. Higher groundwater levels are expected during wet periods of the year, such as spring. These seasonal fluctuations would not be expected to have any significant impact on the recommendations contained in subsequent sections.

5.0 DISCUSSION

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements and is subject to the limitations in the “Important Information and Limitations of this Report” attachment which follows the text of this report.

The foundation engineering guidelines presented in this section have been developed in a manner consistent with the procedures outlined in Part 4 of the Ontario Building Code (OBC) for Limit States Design.

5.2 Foundations

The subsurface conditions vary across the overall site.

Within the Phase 1 area, the subsurface conditions generally consist of fill material over 3 metres of weathered silty clay, overlying unweathered silty clay, which are underlain by glacial till.

Within the Phase 2 area, the subsurface conditions generally consist of 2 to 2.5 metres of weathered silty clay, overlying glacial till, with the surface of the shale bedrock at about 3 to 4 metres depth.

5.2.1 Phase 1 Area

The existing surficial fill materials present on this site are not suitable for the support of the footings, or the slab, and should be removed from within the building footprint. The footings should then be founded on/within the weathered silty clay crust or on engineered fill placed on that bearing surface.

The foundation design parameter values (Serviceability Limit States (SLS) and Ultimate Limit States (ULS) resistances) for spread footing foundations at this phase of the site are based on limiting the stress increases on the grey silty clay at depth to an acceptable level so that foundation settlements do not become excessive. Four important parameters in calculating the stress increase on the grey silty clay under the weathered crust are:

- The thickness of the weathered crust below the underside of the footings;
- The size (dimensions) of the footings;
- The amount of surcharge in the vicinity of the foundation due to landscape fill, underslab fill, floor loads, etc; and,
- The effects of groundwater lowering caused by this or other construction.

It is understood that the proposed finished floor slab levels of the Phase 1 buildings will be at about the existing grade.

For frost protection purposes, the exterior footings should be founded at least 1.5 metres below the finished exterior grade, placing the exterior footings for the structures no deeper than about elevation 84.3 metres. The floor loading for the structures is understood not to exceed 5 kilopascals.

Based on the above elevations and floor loadings, the SLS net bearing resistance and the factored ULS bearing resistance values for spread footing foundations (for buildings and retaining walls) may be taken as follows:

Building Footing Type	Minimum Founding Elevation (metres)	Footing Width or Size (metres)	Net Bearing Resistance at SLS (kPa)	Factored Bearing Resistance at ULS (kPa)
Temporary Office Building Strip Footing	84.3	< 1.0	125	165
Temporary Office Building Pad Footing	84.3	< 1.0	150	165
General Storage Building Strip Footing	84.3	< 1.0	95	165
General Storage Building Pad Footing	84.3	< 1.0	150	165

The ULS bearing resistances presented above have been determined in order limit the potential for overall shear failure of the silty clay. The SLS bearing resistances presented above have been determined in order to limit the final effective stress below the foundations to a value less than the estimated pre-consolidation pressure. These bearing resistances correspond to a settlement resulting from consolidation of the silty clay and are based on the thickness of the weathered crust, empirical correlation between undrained shear strength and pre-consolidation pressure (estimated to be between 90 and 260 kPa based on current vane testing) and experience with similar soils.

For larger footings, footings placed at greater depth, increases in floor loading or increases in exterior grade levels, the above design parameters will change and new values must be calculated taking any such changes into account.

The post construction total and differential settlements of footings sized using the above SLS net bearing resistance values should be less than about 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction.

Consolidation of the silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance values given above should be the full dead load plus sustained live load. The factored dead plus full factored live load should be used in conjunction with the ULS factored bearing resistance.

5.2.2 Phase 2 Area

Grade raises of up to 3 m are acceptable on the Phase 2 area of the site and the foundations guidance below has been developed on that basis.

The existing surficial fill materials and the disturbed silty clay (at borehole 11-4) present on this site are not suitable for the support of the footings, or the slab, and should be removed from within the building footprint.

It is considered that the footings could be founded on/within the weathered silty clay crust or on engineered fill placed on that bearing surface.

The net bearing resistance at Serviceability Limit States (SLS) for pad footings up to 3.0 metres square and for strip footings up to 3.0 metres in width, may be taken as 125 kilopascals. The factored bearing resistance at Ultimate Limit States (ULS) may be taken as 165 kilopascals.

The post construction total and differential settlements of footings sized using the above SLS net bearing resistance values should be less than about 25 and 15 millimetres, respectively, provided that the soil at or below founding level is not disturbed during construction. Further, these bearing resistances correspond to a settlement resulting from consolidation of the silty clay and are based on the thickness of the weathered crust, empirical correlation between undrained shear strength and pre-consolidation pressure and experience with similar soils.

Consolidation of the silty clay is a process which takes months or longer and, as such, results from sustained loading. Therefore, the foundation loads to be used in conjunction with the SLS resistance values given above should be the full dead load plus <u>sustained</u> live load. The factored dead plus <u>full</u> factored live load should be used in conjunction with the ULS factored bearing resistance.

The underside of both the perimeter and interior footings for the building and canopy may be above the surface of the native soils. In addition, when the existing buildings (house, garage, etc) are demolished, the existing foundations and backfill must be removed from within the zone of influence of the new foundations and floor slabs. The zone of influence is considered to extend out and down from the edge of the new footings and edge of slabs at a slope of 1 horizontal to 1 vertical. Where the site preparation leaves the native subgrade level below the proposed underside of footing level, the grade should be raised, within the zone of influence, with Ontario Provincial Standard Specification (OPSS) Granular B Type II placed in maximum 300 millimetre thick lifts and compacted to at least 98 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The same foundation design parameters can be used for this design option, as given above.

At locations where the footings are founded on the weathered silty clay, the short-term shear resistance within the silty clay should be checked using a factored shear strength (Su) value of 40 kPa. The lateral resistance to long term loading of footings on weathered silty clay may be evaluated using a factored tan δ^* lateral sliding resistance value of 0.34.

Where foundations will be supported on engineered fill, a factored tan δ^* lateral sliding resistance value of 0.40 may be used at the base of footing – engineered fill interface.

5.3 Groundwater Management

Based on the design details provided, it is anticipated the underside of foundations will be at about elevation 85.6. The groundwater levels at the site were indicated to be at about elevations ranging from 84.4 to 85.6 m. Based on the underside of footing elevations and the measured groundwater levels, the building excavation inverts may extend to the maximum measured groundwater levels, depending on the time of year, and any dewatering required should be manageable by pumping from sumps within the excavations.

The base of the stormwater management pond (dry retention area) is indicated to be at about 85.1 m (i.e., about 0.5 m below the highest measured groundwater level). Pumping from sumps should also be feasible for groundwater management but higher inflows may be expected depending on the groundwater level at the time of construction. Surface water inflows from precipitation events will also add to the pumping requirements. Ideally excavations would be planned for drier periods, such as summer.

Consideration should be given to carrying out further hydrogeolological assessments to assess the potential risks associated with construction when the facility design is finalized.

Construction Water takings in excess of 50 m^3/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400 m^3/day qualify for self-registration on the MECP's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking less than 400 m^3/day and a Section 53 approval for discharge of water to the environment. A "Water Taking Plan" and a "Discharge Plan" are required by the MECP if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan. A Category 3 PTTW would be required for water takings in excess of 400 m^3/day. The construction water taking permit and registration should be prepared adequately in advance of site excavation works so as not to unduly affect the construction schedule.

5.4 Seismic Site Response Classification

The seismic design provisions of the Ontario Building Code depend, in part, on the shear wave velocity, undrained shear strength or SPT N values of the upper 30 metres of soil and/or rock below founding level. Due to the differing soil conditions across the site, the site class has been evaluated for each of the three proposed buildings.

For design purposes, the proposed Phase 1 temporary office building and general storage building can be assigned a Site Class D.

The Phase 2 permanent office building can be assigned a Site Class C for design.

The glacial till soils and the native silty clay at this site are not considered to be susceptible to liquefaction or cyclic softening in response to the design seismic event.

5.5 Slab on Grade

Conventional slab on grade construction can be used for the structures on this site.

However, for predictable performance of the floor slabs, the existing topsoil, fill materials, and disturbed clay should be removed from within the proposed building areas. Provision should be made for at least 150 millimetres of Ontario Provincial Standard Specification (OPSS) Granular A to form the base for the floor slabs. Any bulk fill required to raise the grade to the underside of the Granular A should consist of OPSS Granular B Type II. The underslab fill should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 98 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

It is understood that the slabs for the building and for support of the fire water tanks will be point loaded and for structural analysis of the slab deflections a modulus of subgrade reaction, ks, is required. It should be noted however that the modulus of subgrade reaction is not a fundamental soil property and its value depends, in part, on the size and shape of the loaded area. For the analysis of the contact stress distribution beneath a raft foundation, its value would depend on the size of the areas over which increased/concentrated contact stresses are anticipated (analogous to equivalent footings beneath the walls and columns) and the size of these areas is in turn related to the value the modulus of subgrade reaction, i.e., they are inter-related. Accordingly, the analysis of the raft slabs should ideally involve an iterative analysis between the determination of the contact stress distribution by the structural engineer and the geotechnical determination of the modulus of subgrade reaction value, until the two are consistent with each other.

For a 0.3 metre by 0.3 metre section of the slab supported on the native weathered silty clay, the modulus of subgrade reaction may be assumed to be in the range of 10 to 30 megapascals per metre. The structural design of the slab at any location should be determined based on whichever value causes the larger effect, since the maximum and minimum values may govern for different locations and load effects.

5.6 Frost Protection

The soils at this site are considered to be frost susceptible. Therefore, all exterior foundation elements should be provided with a minimum of 1.5 metres of earth cover for frost protection purposes. Isolated, unheated footings adjacent to surfaces which are cleared of snow cover during winter months should be provided with a minimum of 1.8 metres of earth cover.

Insulation of the bearing surface with high density polystyrene rigid foam insulation could be considered as an alternative to earth cover for frost protection. The details for footing insulation could be provided if and when required.

Insulation will likely be required at the loading dock, unless the retaining wall footings can be founded at least 1.8 m below the ramp surface (i.e., below the underside of the building foundations). The footings for the retaining walls at the ramp should be provided with insulation, at least 50 mm in thickness, at the underside extending a distance of 1.8 m, less the depth of earth cover, beyond the edge of the footings.

In preparation for the insulation, a levelling mat consisting of 25 millimetres of concrete/mortar sand or 50 millimetres of lean concrete should be placed on the approved bearing surface. Care must be taken to ensure that the insulation is not damaged during construction. Joints should be carefully lap jointed and glued where and if possible. Footings may then be constructed on the surface of the insulation. The type of insulations should be selected such that the bearing pressure on the insulation placed under the footings does not exceed about 35 percent of the insulation’s quoted compressive strength. This is due to the time dependant creep characteristics of this material. For example, the allowable bearing pressures for several strengths of insulation are:

Insulation Type	SLS Resistance (kilopascals)	ULS Factored Resistance (kilopascals)
Dow SM	65	100
Dow Highload 40	90	135
Dow Highload 60	145	205
Dow Highload 100	240	340

To reduce the potential for differential frost heaving across the loading dock ramp, the insulation below the ramp should extend from retaining wall to retaining wall (i.e., across the full width of the ramp).

The insulation which projects beyond the edge of the footings can consist of Dow SM or equivalent, except beneath pavements where HI 60 should be used beyond the footing.

In addition, the building foundations should also be insulated at the loading dock (unless founded 1.8 m below the ramp pavement surface).

A transition detail may be required at the top of the loading dock ramp, where the insulation ends, depending if the footings are maintained at the same elevation or steeped as the ramp grade rises. Further details can be provided as the design progresses.

5.7 Foundation Wall Backfill

The soils at this site are frost susceptible and should not be used as backfill against exterior or unheated foundation elements. To avoid problems with frost adhesion and heaving, these foundation elements should be backfilled with non-frost susceptible sand, or sand and gravel conforming to the requirements for OPSS Granular B Type I.

In areas where pavement or other hard surfacing will abut the building, differential frost heaving could occur between the granular fill and other areas, particularly where clay is present. To control this differential heaving, the backfill adjacent to the foundation wall should be placed to form a frost taper. The frost taper should be brought up to pavement subgrade level from 1.5 metres below finished exterior grade at a slope of 3 horizontal to 1 vertical, or flatter, away from the wall. The granular fill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

It is understood that the native subgrade at or below foundation depth will be sloped away from the foundations at a grade of at least 1% and that the backfill within the building and covered vehicle storage area will consist of free draining OPSS Granular A or Granular B Type II. Considering the planned filling on site and the maximum groundwater levels recorded, foundation drainage is not considered to be required.

5.8 Site Servicing

Excavation for the installation of the site services will generally be through topsoil, fill, weathered silty clay, and possibly into the glacial till.

No unusual problems are anticipated in excavating in the overburden materials using conventional hydraulic excavating equipment, recognizing that boulders may be encountered within the glacial till. Boulders larger than 0.3 metres in size should be removed from the excavation side slopes for worker safety.

The Occupational Health and Safety Act (OHSA) of Ontario indicates that side slopes could be sloped at a minimum of 1 horizontal to 1 vertical (i.e., Type 3 soils).

Some groundwater inflow into the excavations should be expected. However, it should be possible to handle the groundwater inflow by pumping from well filtered sumps established in the floor of the excavations.

At least 150 millimetres of OPSS Granular A should be used as pipe bedding for sewer and water pipes. Where unavoidable disturbance to the subgrade surface does occur, it may be necessary to place a sub-bedding layer consisting of 300 millimetres of compacted OPSS Granular B Type II beneath the Granular A or to thicken the Granular A bedding. The bedding material should in all cases extend to the spring line of the pipe and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density. The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the sandy backfill materials could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

Cover material, from the spring line of the pipe to at least 300 millimetres above the top of pipe, should consist of OPSS Granular A or Granular B Type I with a maximum particle size of 25 millimetres. The cover material should be compacted to at least 95 percent of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the grey brown silty clay and glacial till as trench backfill. Where the trench will be covered with hard surfaced areas, the type of native material placed in the frost zone (between subgrade level and 1.8 metres depth) should match the soil exposed on the trench walls for frost heave compatibility. Trench backfill should be placed in maximum 300 millimetre thick lifts and should be compacted to at least 95 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment.

5.9 Slope Stability

It is understood that retaining walls, potentially up to about 1.2 m in exposed height, will be required at the loading dock. The retaining walls were evaluated using the GeoStudio 2021 Slope/W software for limit equilibrium analysis.

The subsurface stratigraphy used in the analyses was based on the subsurface conditions encountered in Borehole 11-4, which was advanced in relatively close proximity to the proposed loading dock. Input parameters for the analysis are provided in Table 1.

The interpreted subsurface conditions consist of general earth fill, engineered fill (anticipated to replace a surficial layer of topsoil and to raise the founding surface to the underside of footings, if required), overlying a deposit of stiff to very stiff silty clay weathered crust, over glacial till and bedrock.

Table 1: Geotechnical Design Parameters for Stability Analysis

Soil Type	Bulk Unit Weight, γ (kN/m^3)	Shear Strength Parameters		
		Undrained Shear Strength, Su (kPa)	Effective Angle of Internal Friction, f' (°)	Effective Cohesion, c' (kPa)
Earth (Grade Raise) Fill	20	N/A	30	0
Engineered Fill	21.5	N/A	34	0
Weathered Silty Clay	17.5	60	35	5
Glacial Till	21	0	34	0
Bedrock	Impenetrable			

The following conditions were also assumed in the analysis:

- The ground behind the wall will be level.
- Site Class C Seismic site classification, (2022 Geotechnical Investigation report).
- A seismic horizontal loading of 0.201, equal to ½ of the site adjusted PGA value (0.402g for Site Class C).

- A static long term groundwater level of 85.0 m.

With appropriate subgrade preparation and proper placement of earth or granular soils, the up to 1.2 m high cast in place concrete retaining wall, will have a factor of safety greater than 1.5 against deep seated slope instability and a factor of safety greater than 1.1 against seismic global instability for both the existing conditions as well as an assumed condition with a higher groundwater level (at the ground surface). The results of the slope stability analysis are shown on Figures 1 through 4 in Appendix D.

It also understood that the storm water management pond will have side slopes less than 1 m in height with side slopes no steeper than 3 horizontal to 1 vertical. The pond side slopes will have factors of safety of greater than 1.5 or 1.1 against static and seismic instability. The pond side slopes should be provided with erosion control measures (e.g., rip rap) to reduce the potential for sloughing and ravelling of the sideslopes.

5.10 Pavement

In preparation for pavement construction, all topsoil and other unsuitable fill (i.e., fills containing organic matter) should be excavated from the pavement areas.

Those portions of the fill material not containing organic matter may be left in place provided that some long term settlement of the pavement surface can be tolerated. However, the surface of the fill material at subgrade level should be proof rolled with a heavy smooth drum roller under the supervision of qualified geotechnical personnel to compact the surface of the existing fill and to identify soft areas requiring sub-excavation and replacement with more suitable fill.

Sections requiring grade raising to proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow or OPSS Select Subgrade Material. These materials should be placed in maximum 300-millimetre thick lifts and should be compacted to at least 95 percent of the material’s standard Proctor maximum dry density using suitable compaction equipment.

The surface of the subgrade or fill should be crowned to promote drainage of the pavement granular structure. Perforated pipe subdrains should be provided at subgrade level extending from the catch basins for a distance of at least 3 metres in four orthogonal directions, or longitudinally where parallel to a curb.

The pavement structure for car parking areas should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	50 150 450

The pavement structure for access roadways and truck traffic areas should consist of:

Pavement Component	Thickness (millimetres)
Asphaltic Concrete OPSS Granular A Base OPSS Granular B Type II Subbase	90 150 450

The pavement structure for unpaved access roadways and truck traffic areas should consist of:

Pavement Component	Thickness (millimetres)
OPSS Granular A Base OPSS Granular B Type II Subbase	250 450

The granular base and subbase materials should be uniformly compacted to at least 100 percent of the material's standard Proctor maximum dry density using suitable vibratory compaction equipment. The asphaltic concrete should be compacted in accordance with Table 9 of OPSS 310.

The composition of the asphaltic concrete pavement in car parking areas should be as follows:

Superpave 12.5 or HL 3 Surface Course – 50 millimetres

The composition of the asphaltic concrete pavement in access roadways and truck traffic areas should be as follows:

Superpave 12.5 or HL 3 Surface Course – 40 millimetres

Superpave 19.0 or HL 8 Binder Course – 50 millimetres

The above pavement designs are based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the trench backfill and grade raise fill have been adequately compacted to the required density and the subgrade surface not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase and/or to place a woven geotextile beneath the granular materials.

5.11 Corrosion and Cement Type

One sample of soil from borehole 11-5 was submitted to EXOVA Accutest Laboratories Ltd. for chemical analysis related to potential corrosion of exposed buried steel and concrete elements (corrosion and sulphate attack). The results of this testing are provided in Appendix B.

The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures. The results also indicate a high potential for corrosion of exposed ferrous metal.

5.12 Material Reuse

It is understood that excavated materials from the site are to be re-used on site as much as possible. In general, the excavated weathered silty clay and glacial till may be re-used in pavement and landscaped areas. Re-use of the material will depend on the water content of the excavated material. Material that is wetter than optimum will need to be stockpiled and possibly spread to dry prior to re-use. Excavation during wetter times of year should be avoided. Any organics, such as topsoil, should be stripped and saved for re-use in landscaped areas.

The glacial till will likely be wetter than optimum and it should be planned to place the glacial till in landscaped areas. The glacial till should be placed in maximum 0.3 m thick lifts and compacted using a 15 tonne roller compactor in non-vibratory mode to 95% of the materials maximum standard Proctor dry density, if achievable.

The weathered silty clay should placed in maximum 0.3 m thick lifts and compacted using a 15 tonne sheepsfoot compactor in non-vibratory mode to 95% or 98% of the materials maximum standard Proctor dry density in landscaped areas or beneath paved areas, respectively. The surface of the clay should be compacted using a 15 tonne smooth drum roller compactor in non-vibratory mode prior to placement of granular materials.

Ideally, the clay fill should be allowed to sit for 2 to 4 weeks and should be proofrolled after that period prior to the placement of granulars for pavements. Consideration should be given to using a geogrid within the pavement subbase granulars in the pavement structure in areas constructed on clay fill. Delaying final paving of the parking area, for as long as feasible, should be considered as well.

Site excavated materials should be approved by a geotechnical professional prior to placement and prior to placement of pavement granulars.

5.13 Trees

The silty clay deposit that is present at the site is highly sensitive to water depletion by trees of high-water demand during periods of dry weather. When trees draw water from clayey soils, the clay undergoes shrinkage which can result in settlement of adjacent structures. The zone of influence of a tree is considered to be approximately equal to the full mature height of the tree. Therefore, in this area, trees which have a high-water demand should not be planted closer to structures than the ultimate height of the trees. Table 2 provides a list of the common trees in decreasing order of water demand and, accordingly, decreasing risk of potential effects on structures.

It is understood that no trees will be planted in the Phase 1 area of the development. In Phase 2, trees will be planted in front of the building (i.e., on the street side of the building). Based on the current landscaping plan, the trees will be at least 12 m from the foundations walls, and this set back distance will meet the current City guidelines for trees on sensitive marine soils (i.e., reduced set backs from the guidelines will not be required).

It should also be noted that the foundation depths for the proposed building are less than the required 2.1 metres in the current City guidelines and reduced set back distances for tree planting will not be feasible, should the landscaping plan change.

6.0 CLOSURE

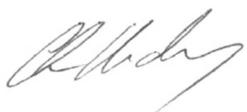
The soils at this site are sensitive to disturbance from ponded water, construction traffic and frost.

All footing and subgrade areas should be inspected by experienced geotechnical personnel prior to filling to establish that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to confirm that the materials used conform to the specifications from both a grading and compaction view point.

Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

Signature Page

WSP Canada Inc.



Chris Hendry, M.Eng., P.Eng.
Senior Geotechnical Engineer

CH/WC/hdw/ml/ljv

https://wsponline-my.sharepoint.com/personal/liz_jones-villeneuve_wsp_com/documents/documents/for ana/21493887 r rev1 updated proposed hydro one facility 2023'05'04.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to WSP by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP cannot be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. WSP should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for WSP to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that WSP be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

TABLE 1

RECORD OF TEST PITS

Test Pit Number (Elevation)	Depth (metres)	Description
11-1 (85.74 metres)	0.00 – 0.15	TOPSOIL
	0.15 – 2.00	Grey brown SILTY CLAY (Weathered Crust)
	2.00	END OF TEST PIT
		Note: Groundwater seepage at 2.00 metres depth

Sample	Depth (m)
1	1.00

Test Pit Number (Elevation)	Depth (metres)	Description
11-2 (85.72 metres)	0.00 – 0.15	TOPSOIL
	0.15 – 1.00	Grey brown SILTY CLAY (Weathered Crust)
	1.00 – 2.40	Grey brown SILTY SAND, some gravel and clay, with cobbles and boulders (GLACIAL TILL)
	2.40	END OF TEST PIT
		Note: Groundwater seepage at 2.00 metres depth

Sample	Depth (m)
1	2.00

Test Pit Number (Elevation)	Depth (metres)	Description
11-3 (85.90 metres)	0.00 – 0.15	TOPSOIL
	0.15 – 2.00	Grey brown SILTY CLAY (Weathered Crust)
	2.00	END OF TEST PIT
		Note: Groundwater seepage at 2.00 metres depth
11-4 (85.08 metres)	0.00 – 0.15	TOPSOIL
	0.15 – 1.60	Grey brown SILTY CLAY (Weathered Crust)
	1.60	END OF TEST PIT
		Note: Groundwater seepage at 1.50 metres depth

Sample	Depth (m)
1	0.50
2	1.00

RECORD OF TEST PIT 11-5

Test Pit Number (Elevation)	Depth (metres)	Description
11-5 (±85.9 metres)	0.00 – 0.27	TOPSOIL
	0.27 – 2.00	Very stiff grey brown SILTY CLAY (Weathered Crust)
		- Field vane test at 0.9 metres > 100 kilopascals
	2.00 – 2.45	Very stiff grey SILTY CLAY
		- Field vane test at 2.1 metres > 100 kilopascals
	2.45 – 2.60	Grey SILTY SAND, some gravel, with cobbles and boulders (GLACIAL TILL)
	2.60	END OF TEST PIT
		Note: Groundwater seepage at 0.9 metres depth

Sample	Depth (m)
1	0.9
2	2.3
3	2.5

TABLE 2

SOME COMMON TREES
IN DECREASING ORDER OF WATER DEMAND

Broad Leaved Deciduous

Poplar

Alder

Aspen

Willow

Elm

Maple

Birch

Ash

Beech

Oak

Deciduous Conifer

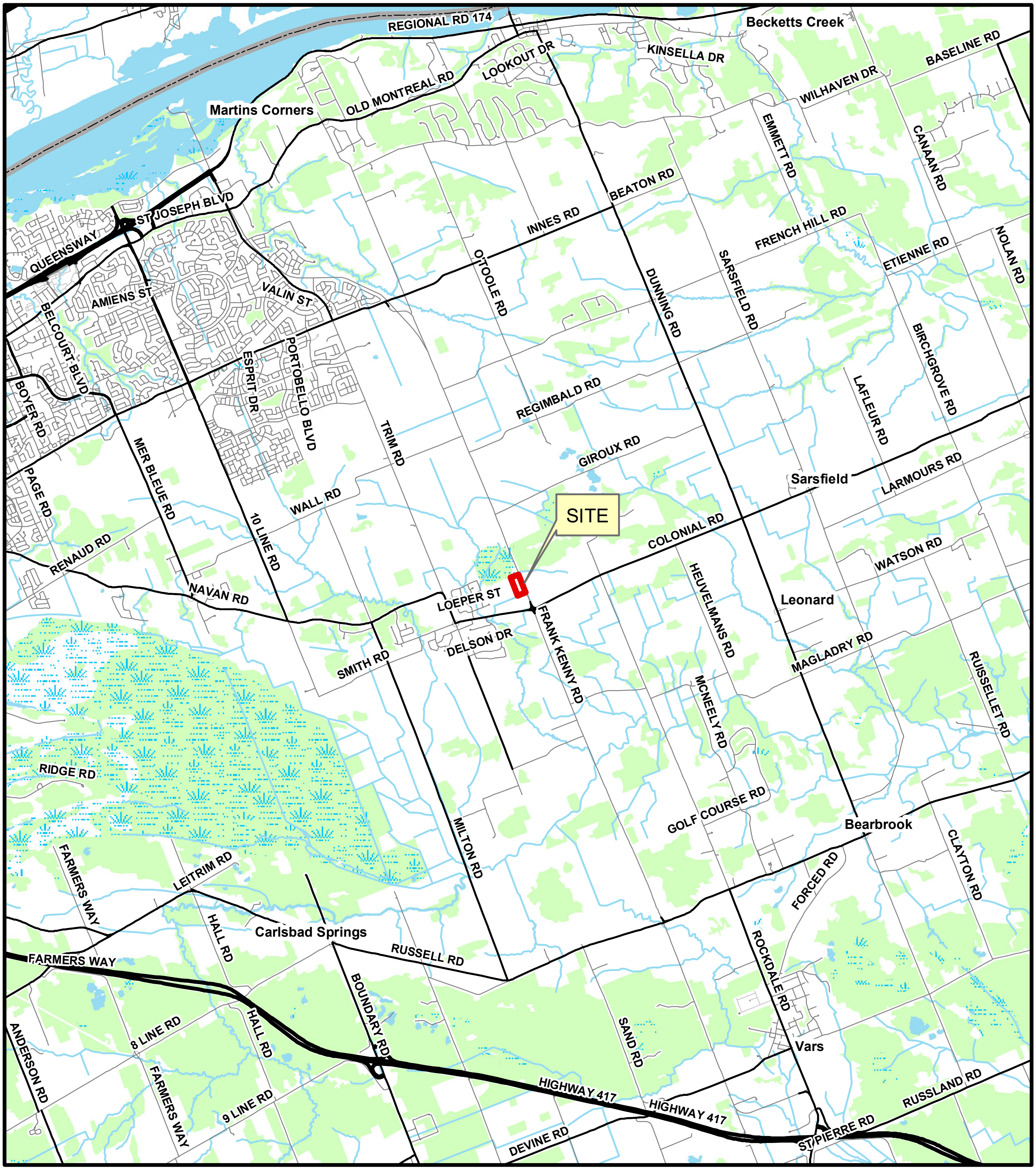
Larch

Evergreen Conifers

Spruce

Fir

Pine



NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT NO. 11-1122-0129-2000

REFERENCE

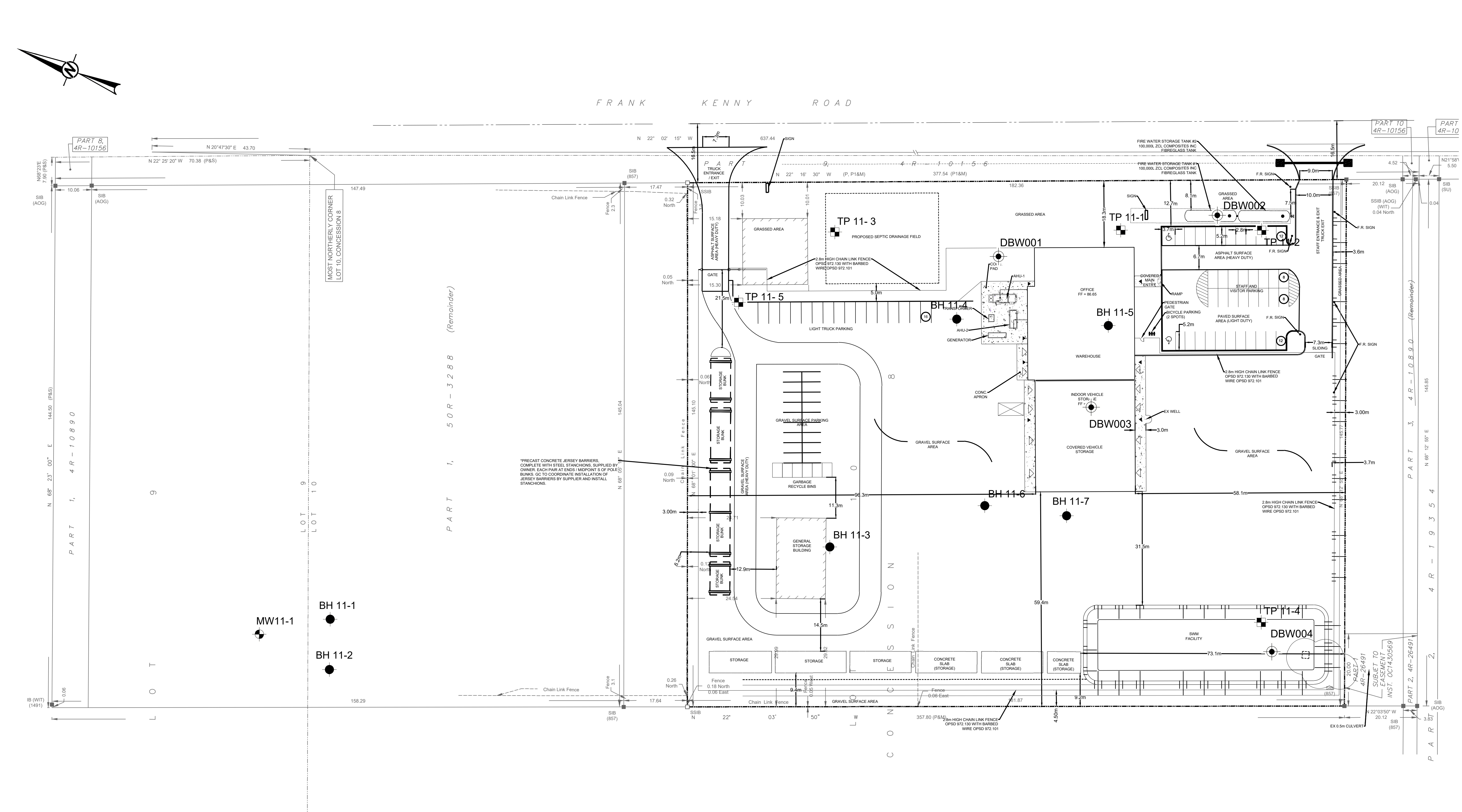
DIGITAL BASE MAP DATA SUPPLIED BY DMTI SPATIAL INC. CANMAP, 2008
PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: MTM ZONE 9

2,500 0 2,500
SCALE 1:100,000 METRES

Golder Associates
Ottawa, Ontario

DATE	Nov. 2011	TITLE	KEY PLAN
DESIGN	BGS		
GIS	BJ		
CHECK	BGS	PROJECT	PROPOSED HYDRO ONE OPERATIONS FACILITY 3406-3450 FRANK KENNY ROAD, OTTAWA, ONTARIO
REVIEW	DHP		FIGURE 1

PROJECT No. 11-1122-0129-2000
SCALE AS SHOWN REV. 0



LEGEND

- APPROXIMATE MONITORING WELL LOCATION, GHD INVESTIGATION
- APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION
- APPROXIMATE MONITORING WELL LOCATION, PREVIOUS INVESTIGATION

REFERENCE(S)

1. BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY J.L. RICHARDS AND ASSOCIATES, DATED MARCH 23, 2022.

DRAFT

0 25 50
1:1,000 METRES

CLIENT
HYDRO ONE

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2022-06-20
DESIGNED	----
PREPARED	ZS/JEM
REVIEWED	----
APPROVED	----

PROJECT
GEOTECHNICAL INVESTIGATION PROPOSED HYDRO ONE OPERATIONS FACILITY 3406-3450 FRANK KENNY ROAD, OTTAWA, ONTARIO

TITLE
SITE PLAN

PROJECT NO.	CONTROL	REV.	FIGURE
21493887	0001	A	1

25 mm IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

Path: \\golder.gds\gal\ottawa\Clients\HydroOne\3406-3450_FrankKennyRd_Ottawa\99_PROJ\21493887_JLR_ProposedOperationsFacility\40_PROD\0001_Geotech\ | File Name: 21493887-0001-BG-0001.dwg | Last Edited By: jmckenzie Date: 2022-06-21 Time:10:19:33 AM | Printed By: JMckenzie Date: 2022-06-21 Time:10:19:50 AM

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. 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.) drive open Sampler for a distance of 300 mm (12 in.)

DD- Diamond Drilling

Dynamic Penetration Resistance; 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, 60^0 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

Peizo-Cone Penetration Test (CPT):

An electronic cone penetrometer with a 60^0 conical tip and a projected 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 (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**C_u or S_u** Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_1	liquid limit
C	consolidaiton (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
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	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

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
F	factor of safety
V	volume
W	weight

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 stresses (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 ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_1	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_1-w_p)
w_s	shrinkage limit
I_L	liquidity index=$(w-w_p)/I_p$
I_c	consistency index=$(w_1-w)/I_p$
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 (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u,s_u	undrained shear strength (ϕ=0 analysis)
p	mean total stress $(\sigma_1+\sigma_3)/2$
p'	mean effective stress $(\sigma'_1+\sigma'_3)/2$
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. τ=c'σ' tan |'
2. Shear strength=(Compressive strength)/2

RECORD OF BOREHOLE: 11-1

PROJECT: 11-1122-0129-2000
LOCATION: See Site Plan
BORING DATE: October 31, 2011
SHEET 1 OF 1
DATUM: Geodetic
SAMPLER HAMMER, 64kg; DROP, 760mm
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	86.36
Dark brown silty fine sand, trace organic matter (FILL)	0.00 / 0.08
Loose brown fine sand (FILL)	86.05 / 0.31
Very stiff to stiff brown to grey brown SILTY CLAY (Weathered Crust)	
Firm grey SILTY CLAY	83.31 / 3.05
End of Borehole	79.40 / 6.96

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	50 DO	6
2	50 DO	12
3	50 DO	5
4	50 DO	3
5	50 DO	1
6	50 DO	WH
7	50 DO	WH

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp —○W— Wl ; 20 40 60 80
ADDITIONAL LAB. TESTING
PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: HEC
CHECKED: SD

MIS-BHS 001 1111220129.GPJ GAL-MIS.GDT 1/12/12 JEM

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: 11-2

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: October 31, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa: nat V. + Q - ●, rem V. ⊕ U - ○ (20, 40, 60, 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		85.78							
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL		0.00							
		Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)		0.08	1	50 DO	12				
1					2	50 DO	7				
					3	50 DO	4				
2		End of Borehole		83.80 1.98							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: 11-3

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: November 1, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diam. (Hollow Stem)	GROUND SURFACE	85.48				
		TOPSOIL	0.00				
		Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	0.15				
1				1	50 DO	5	
2				2	50 DO	2	Native Backfill
				3	50 DO	1	
3		Firm grey SILTY CLAY	82.58 2.90	4	50 DO	WH	
4		Compact to dense dark grey to black SILTY SAND, some gravel, with shale fragments, cobbles, and boulders (GLACIAL TILL)	81.42 4.06				Bentonite Seal; Silica Sand; Standpipe
5				5	50 DO	17	
				6	50 DO	38	W.L. in Standpipe at Elev. 84.38 m on Nov. 14, 2011
		End of Borehole	79.84 5.64				
6							
7							
8							
9							
10							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl; 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

MIS-BHS 001 1111220129.GPJ GAL-MIS.GDT 1/12/12 JEM

RECORD OF BOREHOLE: 11-3A

PROJECT: 11-1122-0129-2000

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: November 2, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
	GROUND SURFACE	85.48			
0	TOPSOIL	0.00			
	Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	0.15			
	Firm grey SILTY CLAY	82.74 2.74			
3			1	73 TP	PH
			2	73 TP	PH
4			3	50 DO	WH
	Compact dark grey to black SILTY SAND, with shale fragments (GLACIAL TILL)	80.73 4.75	4	50 DO	6
5	End of Borehole	80.43 5.05			
	Note: Shallow portion of stratigraphy inferred from BH 11-3				

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp —○W— Wl ; 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: 11-4

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: November 2, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH (m)	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	WATER CONTENT PERCENT
	GROUND SURFACE	85.90					
0	Dark brown clayey topsoil (FILL)	0.00					
	Grey crushed stone (FILL)	0.15					
	Grey brown SILTY CLAY (Disturbed)		1	50 DO	6		
1	Stiff grey brown SILTY CLAY (Weathered Crust)	84.68 / 1.22	2	50 DO	3	rem V. ⊕, nat V. +	○
2						rem V. ⊕, nat V. +	
						rem V. ⊕, nat V. +	
			3	50 DO	16		○
	Compact dark grey to black SILTY SAND, with shale fragments, cobbles, and boulders (GLACIAL TILL)	83.36 / 2.54	4	50 DO	12		
4			5	50 DO	16		
	Highly weathered black SHALE BEDROCK	81.63 / 4.27	6	50 DO	>50		
	End of Borehole Sampler Refusal	81.38 / 4.52					

Column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: 11-5

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: November 1, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	85.64				
0	TOPSOIL	0.00				
	Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	0.15	1	50 DO	9	
1			2	50 DO	7	Native Backfill
2			3	50 DO	2	
	Compact dark grey to black SILTY SAND, some gravel, with cobbles, boulders, and shale fragments (GLACIAL TILL)	83.51 / 2.13	4	50 DO	11	
3			5	50 DO	16	Bentonite Seal; Silica Sand; Standpipe
	Highly weathered black SHALE BEDROCK	81.83 / 3.81	6	50 DO	>50	
4	End of Borehole	81.58 / 4.06				W.L. in Standpipe at Elev. 84.37 m on Nov. 14, 2011

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl ; 20 40 60 80

ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

MIS-BHS 001 1111220129.GPJ GAL-MIS.GDT 1/12/12 JEM

RECORD OF BOREHOLE: 11-6

PROJECT: 11-1122-0129-2000
LOCATION: See Site Plan
BORING DATE: November 1, 2011
SHEET 1 OF 1
DATUM: Geodetic
SAMPLER HAMMER, 64kg; DROP, 760mm
PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m
		GROUND SURFACE	85.44			
0	Power Auger, 200 mm Diam. (Hollow Stem)	TOPSOIL	0.00			
		Very stiff to stiff brown SILTY CLAY (Weathered Crust)	0.15			
1				1	50 DO	4
2				2	50 DO	2
			82.70	3	50 DO	7
		Compact dark grey to black SILTY SAND, with shale fragments (GLACIAL TILL)	2.74			
3			82.39			
		Highly weathered black SHALE BEDROCK	3.05	4	50 DO	21
			81.78			
		End of Borehole	3.66			

Chart columns (no data plotted): DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa, nat V. + Q - ●, rem V. ⊕ U - ○ (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢ W ⊣ Wl (20, 40, 60, 80); ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: HEC
CHECKED: SD

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: 11-7

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: November 1, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m
		GROUND SURFACE	85.42			
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL	0.00			
		Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)	0.15			
1				1	50 DO	5
				2	50 DO	2
2		Compact dark grey to black SILTY SAND, with shale fragments, cobbles, and boulders (GLACIAL TILL)	83.44 1.98	3	50 DO	16
3				4	50 DO	17
		Highly weathered SHALE BEDROCK	81.91 3.51	5	50 DO	20
4				6	50 DO	>50
		End of Borehole	80.70 4.72			
5						
6						
7						
8						
9						
10						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

PROJECT: 11-1122-0129-2000

RECORD OF BOREHOLE: MW11-1

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: October 31, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		85.96							
0	Power Auger, 200 mm Diam. (Hollow Stem)	Grey crushed stone (FILL)		0.00	1	50 DO	22				Gravel
				85.35							
		Very stiff to stiff grey brown SILTY CLAY (Weathered Crust)		0.61	2	50 DO	8				Bentonite Seal
1					3	50 DO	5				Silica Sand
2					4	50 DO	4				
				83.52							
		Firm grey SILTY CLAY		2.44	5	50 DO	1				51 mm Diam. PVC #10 Slot Screen
3					6	50 DO	1				
				82.30							
		End of Borehole		3.66							
4											W.L. in Screen at Elev. 84.77 m on November 4, 2011
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: HEC

CHECKED: SD

APPENDIX B

Results of Basic Chemical Analysis Exova Accutest Report No. 1126218

REPORT OF ANALYSIS

Client: **Golder Associates Ltd. (Ottawa)**
32 Steacie Drive

Kanata, ON
K2K 2A9
Attention: **Mr. Stephen Dunlop**

Report Number: 1126218
Date: 2011-11-15
Date Submitted: 2011-11-08

Project: 11-1122-0129

P.O. Number:
Matrix: Soil

Chain of Custody Number: **127521**

		LAB ID:	923658					**GUIDELINE**		
		Sample Date:	2011-11-01							
		Sample ID:	11-5 Sa2							
PARAMETER	**UNITS**	**MRL**						**TYPE**	**LIMIT**	**UNITS**
Chloride	%	0.002	0.004							
Electrical Conductivity	mS/cm	0.05	0.27							
pH			7.5							
Resistivity	ohm-cm	1	3700							
Sulphate	%	0.01	0.01							

MRL = Method Reporting Limit INC = Incomplete AO = Aesthetic Objective OG = Operational Guideline MAC = Maximum Allowable Concentration IMAC = Interim Maximum Allowable Concentration

Comment:

APPROVAL: ____________________
Lorna Wilson
Inorganic Lab Supervisor

Methods references and/or additional QA/QC information available on request.

APPENDIX C

Stratigraphic and Instrumentation Logs (DBW001 to DBW004) GHD Project Number 12575389

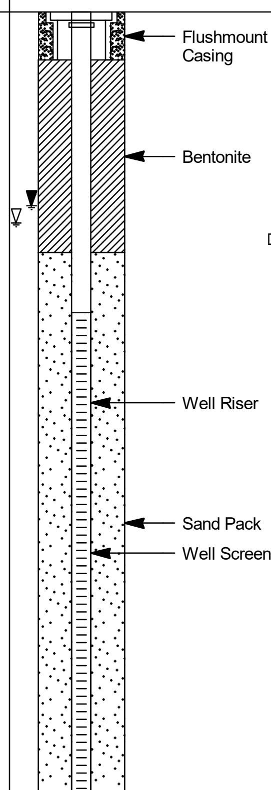
STRATIGRAPHIC AND INSTRUMENTATION LOG (OVERBURDEN)

PROJECT NAME: Hydrogeological Assessment and Soil Quality Investigation - New Orleans OC
PROJECT NUMBER: 12575389
CLIENT: Hydro One Networks Inc.
LOCATION: 3440 Frank Kenny Road, Navan, Ontario

HOLE DESIGNATION: DBW001
DATE COMPLETED: 7 April 2022
DRILLING METHOD: 205mm O.D HSA + Split Spoon
FIELD PERSONNEL: L. McCann

DEPTH m BGS	STRATIGRAPHIC DESCRIPTION & REMARKS	ELEV. m AMSL
	GROUND SURFACE	86.60
	TOP OF RISER	86.47
0.00–0.61	FILL-SAND and GRAVEL; dense; coarse grained; poorly graded; brown to grey; moist	85.99
0.61–1.07	FILL-SANDY SILT; very stiff; low plasticity; brown; dry	85.53
1.07–2.87	CL-CLAY (NATIVE); firm; low plasticity; brown; dry to moist - becomes soft, brown to grey, moist at 2.49m BGS	83.73
2.87–3.96	GP-GC-CLAYEY GRAVEL; compact; medium grained; poorly graded; grey; very wet	82.64
3.96	END OF BOREHOLE @ 3.96m BGS Refusal at 3.96 mBGS	

Monitoring Well



SAMPLE NUMBER	INTERVAL	REC (%)	'N' Value	PID (ppm)
1 (DBW001-1-2)		70	35	0.1
2 (DBW001-3-4/DUP003)		80	17	0.0
3		80	9	0.0
4 (DBW001-7.5-9.5)		60	2	0.1
5		25	27	0.0
6		20	<50	0.0

WELL DETAILS
Screened interval:
85.08 to 82.64m AMSL
1.52 to 3.96m BGS
Length: 2.44m
Diameter: 51mm
Slot Size: 10
Material: PVC
Seal:
86.45 to 82.64m AMSL
0.15 to 3.96m BGS
Material: Bentonite
Sand Pack:
85.38 to 82.64m AMSL
1.22 to 3.96m BGS
Material: #2 Silica Sand

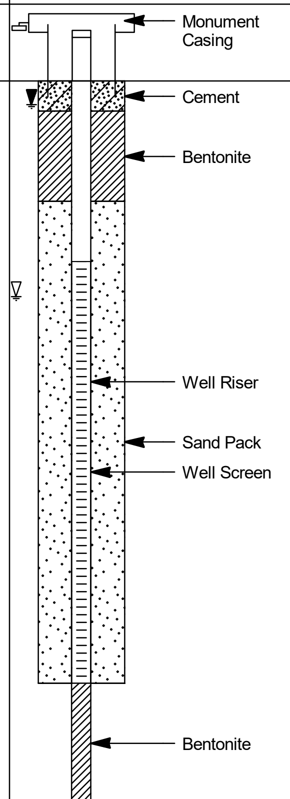
NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE
WATER FOUND ▽ 4/13/2022 STATIC WATER LEVEL ▼ 4/19/2022
CHEMICAL ANALYSIS ◯ GRAIN SIZE ANALYSIS ☐

STRATIGRAPHIC AND INSTRUMENTATION LOG
(OVERBURDEN)

PROJECT NAME: Hydrogeological Assessment and Soil Quality Investigation - New Orleans OC
HOLE DESIGNATION: DBW002
DATE COMPLETED: 6 April 2022
PROJECT NUMBER: 12575389
DRILLING METHOD: 205mm O.D HSA + Split Spoon
CLIENT: Hydro One Networks Inc.
FIELD PERSONNEL: L. McCann
LOCATION: 3440 Frank Kenny Road, Navan, Ontario

DEPTH m BGS	STRATIGRAPHIC DESCRIPTION & REMARKS	ELEV. m AMSL
	TOP OF RISER	86.51
	GROUND SURFACE	85.58
	TOPSOIL-CLAYEY SILT; firm; low plasticity; brown; dry to moist	
	CL-CLAY (NATIVE); firm; low plasticity; brown; dry	85.27
0.5		
1.0	- becomes grey, very stiff, dry to moist from 0.97 to 1.22m BGS	
	CLG-GRAVELLY CLAY; hard; low plasticity; dark brown; wet	84.36
1.5		
2.0		
2.5	- sand lense from 2.59 to 2.64m BGS	
3.0		
3.5		
	END OF BOREHOLE @ 3.66m BGS	81.92
4.0		
4.5		
5.0		
5.5		
6.0		
6.5		

Monitoring Well



WELL DETAILS
Screened interval:
84.66 to 82.53m AMSL
0.91 to 3.05m BGS
Length: 2.13m
Diameter: 51mm
Slot Size: 10
Material: PVC
Seal:
85.43 to 84.97m AMSL
0.15 to 0.61m BGS
Material: Bentonite
Sand Pack:
84.97 to 82.53m AMSL
0.61 to 3.05m BGS
Material: #2 Silica Sand

SAMPLE NUMBER	INTERVAL	REC (%)	'N' Value	PID (ppm)
DBW002-0-1				
1		80	33	0.1
DBW002-2-3				
2		90	9	0.1
3		60	41	0.1
DBW002-5-7				
4		30	40	0.0
5		85	35	0.1

NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE
WATER FOUND ∇ 4/13/2022 STATIC WATER LEVEL ▼ 4/19/2022
CHEMICAL ANALYSIS ◯ GRAIN SIZE ANALYSIS ☐

STRATIGRAPHIC AND INSTRUMENTATION LOG
(OVERBURDEN)

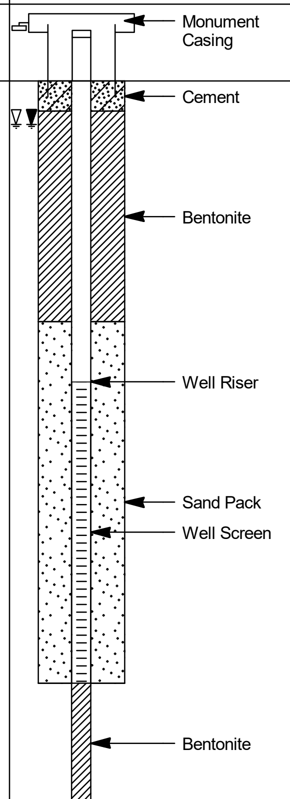
PROJECT NAME: Hydrogeological Assessment and Soil Quality Investigation - New Orleans OC
PROJECT NUMBER: 12575389
CLIENT: Hydro One Networks Inc.
LOCATION: 3440 Frank Kenny Road, Navan, Ontario

HOLE DESIGNATION: DBW003
DATE COMPLETED: 6 April 2022
DRILLING METHOD: 205mm O.D HSA + Split Spoon
FIELD PERSONNEL: L. McCann

DEPTH m BGS	STRATIGRAPHIC DESCRIPTION & REMARKS	ELEV. m AMSL
	TOP OF RISER	86.54
	GROUND SURFACE	85.64
	TOPSOIL-CLAYEY SILT; firm; low plasticity; brown; dry; rootlets	
	CL-CLAY (NATIVE); firm; low plasticity; brown; dry to moist	85.44
1.5	- becomes very soft, brown to grey, moist from 1.52 to 2.13m BGS	
	GP-GC-CLAYEY GRAVEL; compact; medium grained; poorly graded; dark brown; very wet	83.35
	END OF BOREHOLE @ 3.66m BGS	81.98

Depth scale (m BGS): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0, 6.5

Monitoring Well



SAMPLE

NUMBER	INTERVAL	REC (%)	'N' Value	PID (ppm)
DBW003-0-1 1		90	6	0.1
DBW003-2-3 2		80	5	0.0
3		100	0	0.1
4		50	27	0.0
5		25	21	0.0

WELL DETAILS
Screened interval:
84.12 to 82.59m AMSL
1.52 to 3.05m BGS
Length: 1.52m
Diameter: 51mm
Slot Size: 10
Material: PVC
Seal:
85.49 to 84.42m AMSL
0.15 to 1.22m BGS
Material: Bentonite
Sand Pack:
84.42 to 82.59m AMSL
1.22 to 3.05m BGS
Material: #2 Silica Sand

NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE
WATER FOUND ⊻ 4/13/2022 STATIC WATER LEVEL ▼ 4/19/2022
CHEMICAL ANALYSIS

STRATIGRAPHIC AND INSTRUMENTATION LOG (OVERBURDEN)

PROJECT NAME: Hydrogeological Assessment and Soil Quality Investigation - New Orleans OC

HOLE DESIGNATION: DBW004

PROJECT NUMBER: 12575389

DATE COMPLETED: 6 April 2022

CLIENT: Hydro One Networks Inc.

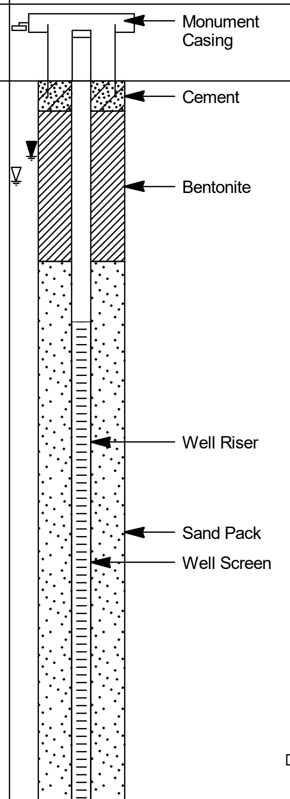
DRILLING METHOD: 205mm O.D HSA + Split Spoon

LOCATION: 3440 Frank Kenny Road, Navan, Ontario

FIELD PERSONNEL: L. McCann

DEPTH m BGS	STRATIGRAPHIC DESCRIPTION & REMARKS	ELEV. m AMSL	SAMPLE NUMBER	INTERVAL	REC (%)	'N' Value	PID (ppm)
	TOP OF RISER	86.10					
	GROUND SURFACE	85.11					
	TOPSOIL-CLAYEY SILT; firm; low plasticity; brown; dry; rootlets	84.91	DBW004-0-1 / 1		100	4	0.2
0.5	CL-CLAY (NATIVE); firm; low plasticity; brown; dry						
1.0	- becomes softer, moist from 1.07 to 1.37m BGS		DBW004-2-3 / 2		100	5	0.0
1.5	- very soft, wet from 1.60 to 3.66m BGS						
2.0			3		100	0	0.1
2.5			4		100	0	0.1
3.0							
3.5			5 / DBW004-10-12		100	0	0.1
	END OF BOREHOLE @ 3.66m BGS	81.45					
4.0							
4.5							
5.0							
5.5							
6.0							
6.5							

Monitoring Well



WELL DETAILS
Screened interval:
83.89 to 81.45m AMSL
1.22 to 3.66m BGS
Length: 2.44m
Diameter: 51mm
Slot Size: 10
Material: PVC
Seal:
84.96 to 84.19m AMSL
0.15 to 0.91m BGS
Material: Bentonite
Sand Pack:
84.19 to 81.45m AMSL
0.91 to 3.66m BGS
Material: #2 Silica Sand

NOTES: MEASURING POINT ELEVATIONS MAY CHANGE; REFER TO CURRENT ELEVATION TABLE
WATER FOUND ∇ 4/13/2022 STATIC WATER LEVEL ▼ 4/19/2022
CHEMICAL ANALYSIS ○ GRAIN SIZE ANALYSIS □

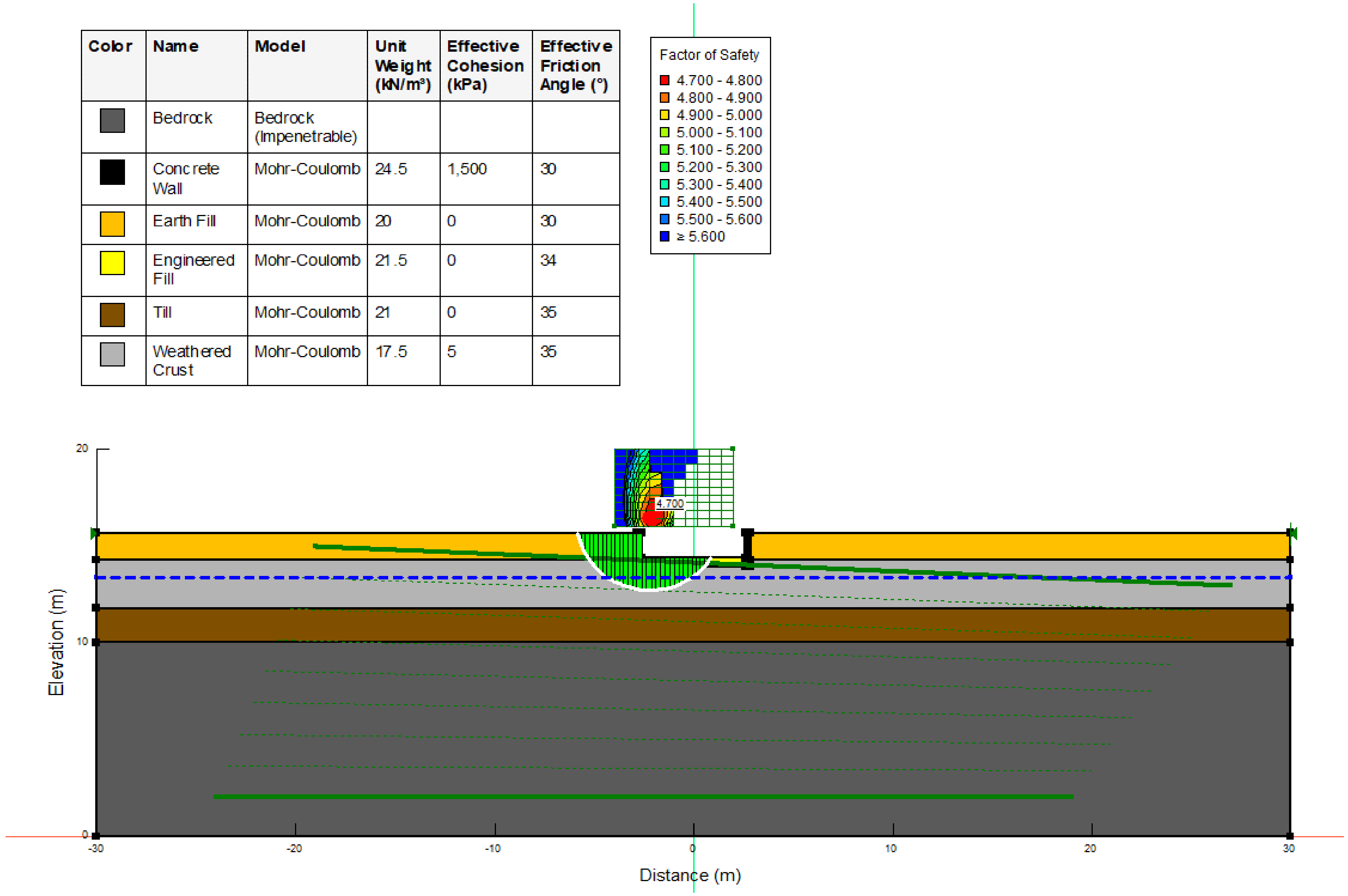
APPENDIX D

Slope Stability Figures

Section AA'

Case 1 – Static Analysis

Color	Name	Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Concrete Wall	Mohr-Coulomb	24.5	1,500	30
	Earth Fill	Mohr-Coulomb	20	0	30
	Engineered Fill	Mohr-Coulomb	21.5	0	34
	Till	Mohr-Coulomb	21	0	35
	Weathered Crust	Mohr-Coulomb	17.5	5	35



Slope Stability Analysis	
D07-12-22-0057-3440 Frank Kenny Road	

WSP GOLDER

Project No:	21490288
Drawn:	KG
Date:	May 20, 2022
Checked:	WC
Review:	WC

FIGURE 1

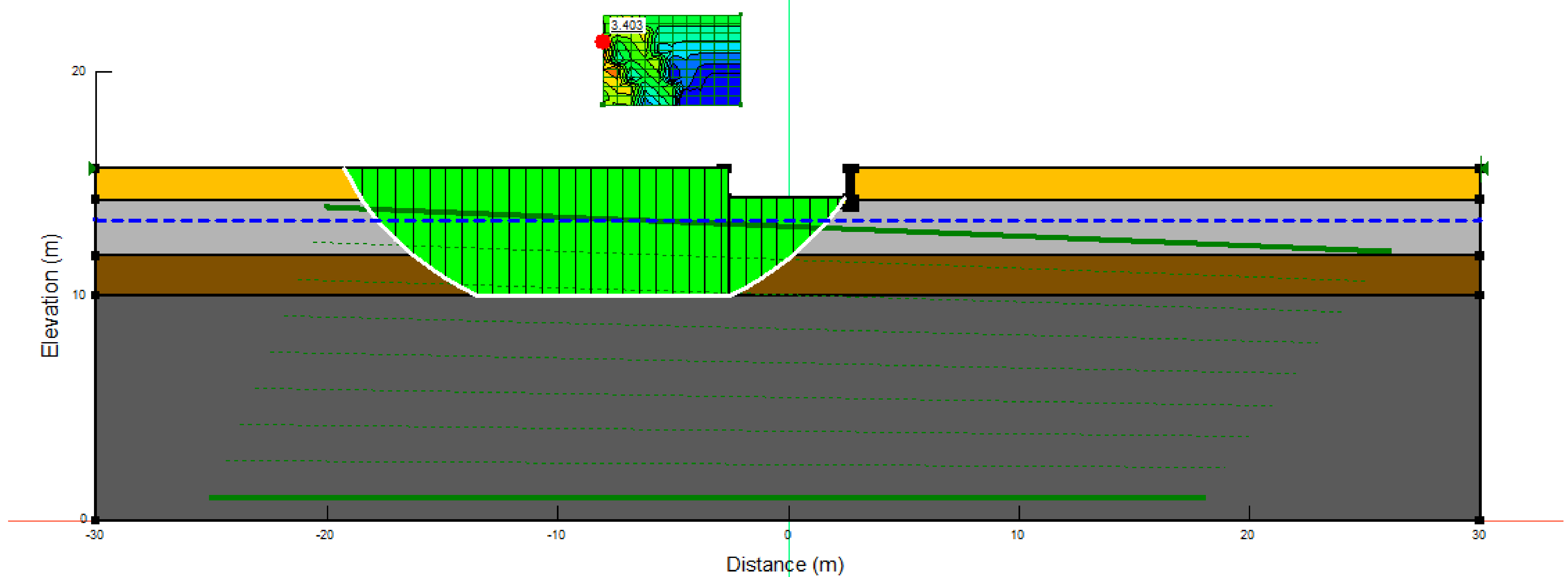
Section AA'

Case 2 – Seismic Analysis

Horz Seismic Coef.: 0.201 g
Vert Seismic Coef.: 0

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Concrete Wall	Mohr-Coulomb	24.5		1,500	30
	Earth Fill	Mohr-Coulomb	20		0	30
	Engineered Fill	Mohr-Coulomb	21.5		0	34
	Till	Mohr-Coulomb	21		0	35
	Weathered Crust	Undrained (Phi=0)	17.5	60		

Factor of Safety
- 3.403 - 3.503
- 3.503 - 3.603
- 3.603 - 3.703
- 3.703 - 3.803
- 3.803 - 3.903
- 3.903 - 4.003
- 4.003 - 4.103
- 4.103 - 4.203
- 4.203 - 4.303
- ≥ 4.303

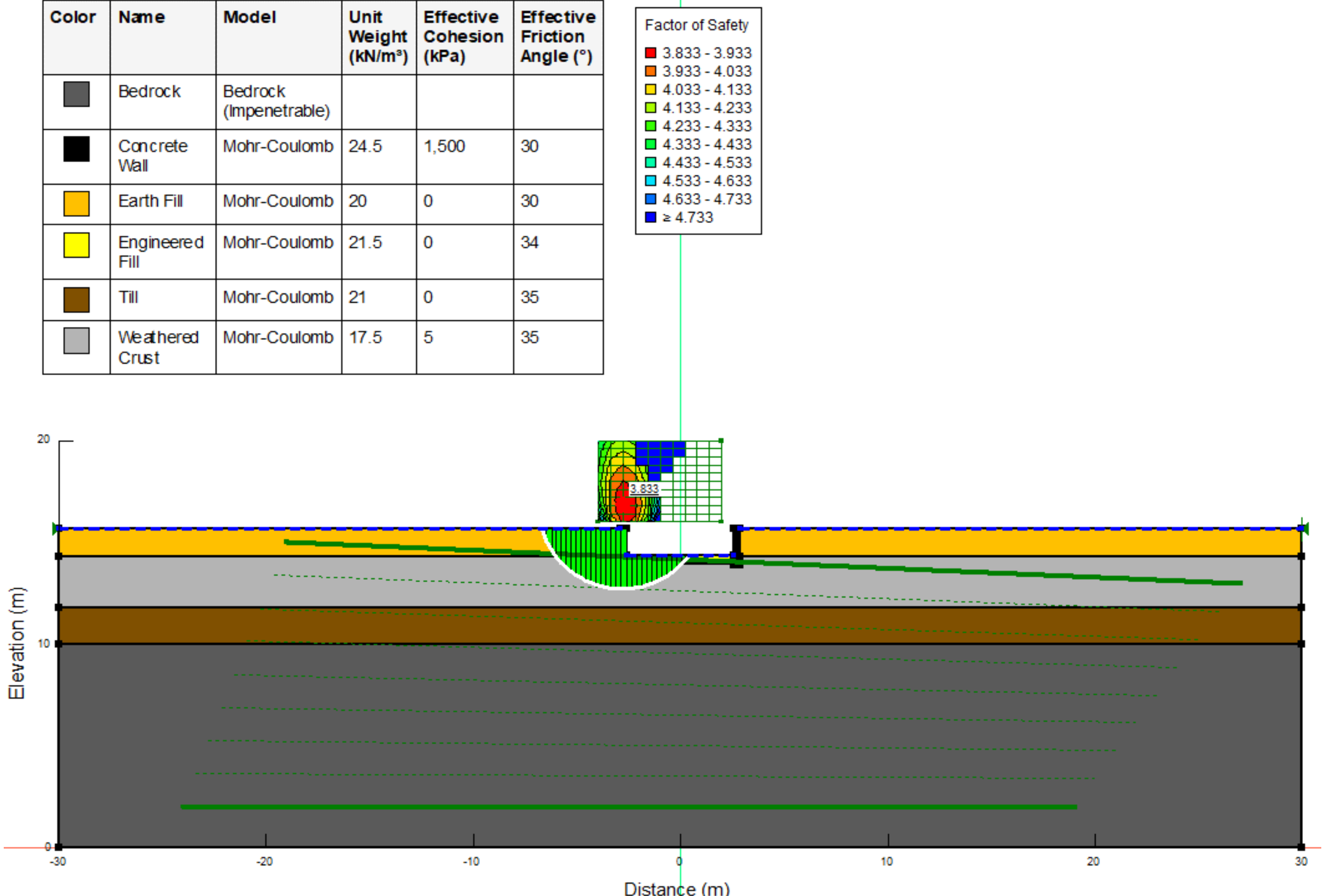


WSP GOLDER	**Slope Stability Analysis** **D07-12-22-0057-3440 Frank Kenny Road**	Project No: 21490288 Drawn: KG Date: May 20, 2022 Checked: WC Review: WC	**FIGURE 2**

Section AA'

Case 1 – Static Analysis

Color	Name	Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)			
	Concrete Wall	Mohr-Coulomb	24.5	1,500	30
	Earth Fill	Mohr-Coulomb	20	0	30
	Engineered Fill	Mohr-Coulomb	21.5	0	34
	Till	Mohr-Coulomb	21	0	35
	Weathered Crust	Mohr-Coulomb	17.5	5	35



WSP GOLDER

Slope Stability Analysis

D07-12-22-0057-3440 Frank Kenny Road

Project No:	21490288
Drawn:	KG
Date:	December 12, 2022
Checked:	CH
Reviewed:	CH

FIGURE 3

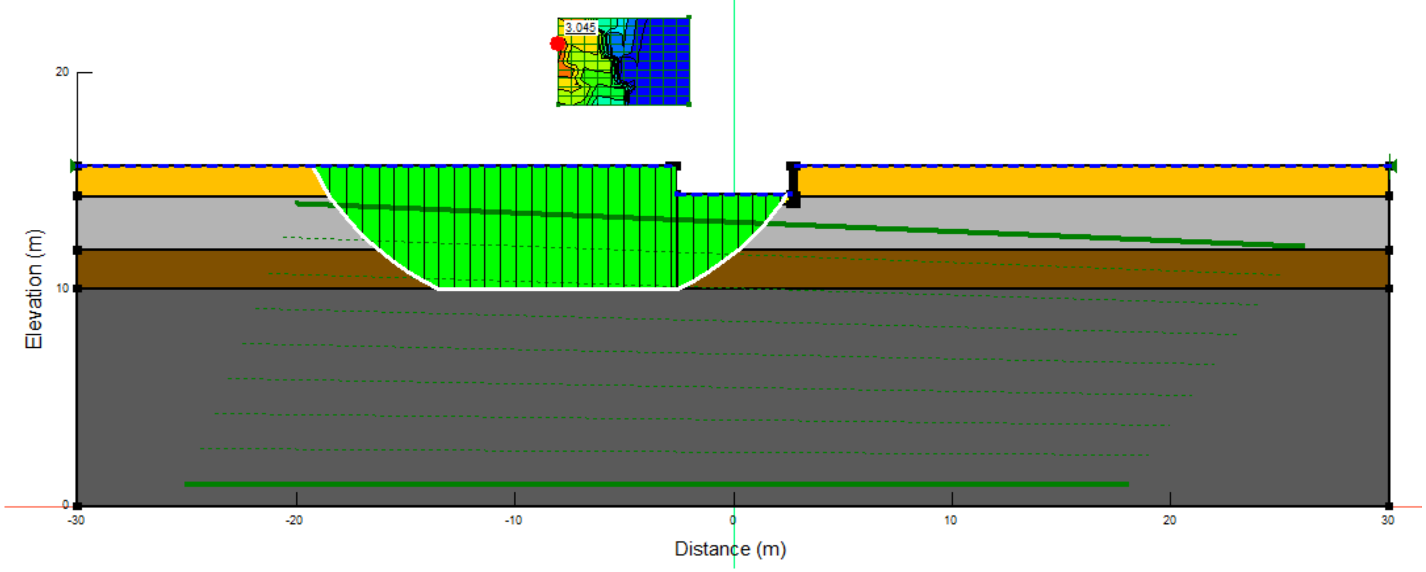
Section AA'

Case 2 – Seismic Analysis

Horz Seismic Coef.: 0.201 g
Vert Seismic Coef.: 0

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bedrock	Bedrock (Impenetrable)				
	Concrete Wall	Mohr-Coulomb	24.5		1,500	30
	Earth Fill	Mohr-Coulomb	20		0	30
	Engineered Fill	Mohr-Coulomb	21.5		0	34
	Till	Mohr-Coulomb	21		0	35
	Weathered Crust	Undrained (Phi=0)	17.5	60		

Factor of Safety
3.045 - 3.145
3.145 - 3.245
3.245 - 3.345
3.345 - 3.445
3.445 - 3.545
3.545 - 3.645
3.645 - 3.745
3.745 - 3.845
3.845 - 3.945
≥ 3.945



WSP GOLDER	**Slope Stability Analysis** **D07-12-22-0057-3440 Frank Kenny Road**	Project No: 21490288 Drawn: KG Date: December 12, 2022 Checked: CH Reviewed: CH	**FIGURE 4**

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