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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 401/Holt Road Underpass Structure
Clarington, Ontario
G.W.P. 2101-08-00

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REPORT

Golder Associates

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/HOLT ROAD INTERCHANGE STRUCTURE
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the Highway 401/Holt Road Interchange reconfiguration in the Town of Clarington, Regional Municipality of Durham, Ontario.

This report addresses the results of the subsurface investigation carried out for the reconstruction/replacement of the Interchange underpass structure.

The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0059 dated March 2009 and associated clarifications, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The existing Highway 401/Holt Road Underpass bridge is located near the entrance to the Darlington Nuclear Power Plant approximately 10 km east of Oshawa, Ontario. According to the original design drawings prepared by Department of Highways – Ontario, dated 1961, the existing four-span underpass structure is about 60 m long with inner span lengths of about 18 m and outer span lengths of about 12 m, and the bridge deck is about 10 m wide. Based on the original design drawings, the existing abutments are supported on piles driven into the very dense native till deposits and the piers are supported on spread footings founded on the native glacial till deposits between about Elevation 108.2 m and 109.4 m.

Based on the preliminary drawings of the new Highway 401/Holt Road Interchange provided by URS (Holt Road – Conceptual Preferred Plan, on September 13, 2012), we understand that the existing bridge will be removed and a new Underpass bridge constructed about 30 m to the east of the existing bridge.

In general, the terrain in the area of the proposed new bridge is relatively flat, with the natural ground surface in the vicinity of the structure site ranging between about Elevation 111 m and 113 m.

The Highway 401 grade in the vicinity of the existing and the new Holt Road Interchange is at about Elevation 112 m. The existing Holt Road Underpass approach embankments consist of earth fill, up to about 6.5 m high, with the Holt Road surface at about Elevation 118.5 m. The existing approach embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V).

2.1 Previous Investigation

The results of a previous geotechnical investigation carried out at the existing Highway 401/Holt Road bridge site were obtained from the MTO GEOCRES library, as summarized in a letter prepared by the Department of Highways – Ontario titled "Darlington Twp. Bridge No. 8, Holt Road Underpass at Highway 401 Intersection, District No. 7", dated March 7, 1961, GEOCRES No. BA851-E.

During the previous investigation, a total of seven (7) boreholes (Borehole Nos. 1 to 7, inclusive) were advanced in the general vicinity of the existing bridge as shown on Drawing 1. A copy of the original borehole logs is included in Appendix C.

In general, the subsoils encountered in the above noted boreholes consist of a surficial deposit of granular fill, 0.3 m to 1.5 m thick, underlain by a 0.3 m to 1.4 m thick layer of topsoil. The topsoil is underlain by a deposit of silty sand till. The silty sand till is described in the borehole logs as gravelly / pebbly. The surface of the silty sand till was encountered between depths of about 0.6 m and 2.1 m below ground surface (between Elevation 111 m and 110 m according to the reference datum used on the borehole logs). The boreholes were terminated within the silty sand till at depths ranging from about 3 m to 9 m below ground surface (Elevation 108 m to 103 m). There were no groundwater levels nor any indication of groundwater being encountered during drilling shown on the borehole logs.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out on November 22, 2012, during which time two boreholes (Boreholes HR-1 and HR-2) were advanced using a track-mounted CME-55 drill rig, supplied and operated by Strong Soil Search Inc. of Claremont, Ontario. Borehole HR-1 was advanced on the east side of the proposed north abutment and Borehole HR-2 was advanced on the west side of the proposed south abutment, approximately at the locations shown on Drawing 1.

The boreholes were drilled using 108 mm diameter solid stem augers to depths of 7.8 m and 6.3 m below ground surface. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586)[1].

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations and are noted on the borehole records contained in Appendix A. The boreholes were backfilled in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. The geotechnical laboratory testing was completed according to MTO procedures and/or ASTM standards as applicable.

The as-drilled borehole locations and ground surface elevations were measured/surveyed in the field relative to temporary benchmarks provided by URS. The borehole locations (referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to geodetic datum) are summarized below and are shown on Drawing 1.

[1] ASTM International, ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
HR-1	4,860,786.9	367,290.0	111.7	7.8
HR-2	4,860,707.2	367,300.7	111.7	6.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2] and *Urban Geology of Canadian Cities* (Karrow and White, 1998)[3]. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. Within the area approximately bounded by Holt Road and Morgan's Road, the surficial glaciolacustrine deposits are absent or of limited thickness and the Bowmanville Till unit is frequently present immediately below the ground surface. Between these limits, an extensive surficial deposit of clayey silt to silty clay is present over the Bowmanville Till (Karrow and White, 1998). More recent alluvial deposits of gravel, sand, silt and/or clay are present in the valleys associated with Bowmanville Creek, Soper Creek, Wilmot Creek and Graham Creek.

The overburden soils are underlain by limestone bedrock of the Lindsay Formation, Simcoe Group (Geological Survey of Canada, 1997).[4]

4.2 Subsurface Conditions

As part of the subsurface investigation, two boreholes were advanced at the proposed new Highway 401/Holt Road Underpass structure site. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The detailed results of geotechnical laboratory testing are also presented on Figures B1 to B3 contained in Appendix B. The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic section on Drawing 1 are inferred from non-continuous 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.

In summary, the subsurface conditions encountered at the site consist of a fill deposit comprised of loose to compact sandy silt between 1.8 m and 2.3 m thick, underlain by a very dense sandy silt to sand and silt till deposit interlayered

[2] Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

[3] Karrow, P. F., and White, O. L., 1998. *Urban Geology of Canadian Cities.* Geological Association of Canada Special Paper No. 42. St. John's, Nfld.

[4] Ontario Geological Society, 1991. *Geology of Ontario*. Special Volume 4, Part 1. Eds. P.C. Thurston, H.R. Williams, R.H. Sutcliffe and G.M. Stott. Ministry of Northern Development and Mines, Ontario.

with clayey silt till. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Sandy Silt to Silty Sand (Fill)

A deposit of sandy silt to silty sand fill was encountered immediately below ground surface in both of the boreholes. The deposit was encountered at Elevation 111.7 m and the thickness of the deposit is 2.3 m and 1.8 m, in Boreholes HR-1 and HR-2, respectively.

The fill consists of sandy silt to silty sand containing trace clay, trace to some gravel and organics and rootlets.

The Standard Penetration Test (SPT) “N” values measured within the fill deposit range from 7 blows to 15 blows per 0.3 m of penetration, indicating a loose to compact relative density

The natural water content measured on three samples of the fill ranges between 11 per cent and 16 per cent.

4.2.2 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in Borehole HR-1 and within the upper portion of the sandy silt to sand and silt till deposit in Borehole HR-2. The surface of the clayey silt till was encountered at a depth of 2.3 m below ground surface, approximately Elevation 109.4 m, in both boreholes. This till deposit is about 1.5 m and 0.6 m thick in Boreholes HR-1 and HR-2, respectively.

The measured SPT “N” values within this deposit range from 28 to 33 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

This glacial till deposit consists of clayey silt with sand to some sand, containing trace to some gravel. The results of grain size distribution tests completed on two selected samples of the clayey silt till are shown on Figure B1 in Appendix B. The grain size distribution for the clayey silt till sample taken from Borehole HR-1 closely resembles the grain size distributions of the sandy silt to sand and silt till, suggesting the clayey silt till layer is likely a transition zone to the underlying more granular till deposit.

Atterberg limits testing was conducted on two selected samples of the clayey till and measured plastic limits of 10 per cent and 15 per cent, liquid limits of 15 per cent and 33 per cent and plasticity indices of approximately 5 per cent and 18 per cent. These test results, which are plotted on a plasticity chart on Figure B2 in Appendix B, confirm that the deposit consists of clayey silt of low plasticity.

The natural water content measured on two samples of the clayey silt till was 10 and 15 per cent.

4.2.3 Sandy Silt to Sand and Silt (Till)

A deposit of sandy silt to sand and silt till was encountered underlying the fill at a depth of 1.8 m below ground surface in Borehole HR-2 and underlying the clayey silt till at a depth of 3.8 m below ground surface in Borehole HR-1, at Elevation 109.9 m and 107.9 m, respectively. Both of the boreholes were terminated within this till deposit at depths of 6.3 m and 7.8 m (Elevation 105.4 m and 103.9 m), in Boreholes HR-2 and HR-1, respectively.

This glacial till deposit consists of sandy silt to sand and silt, containing trace to some clay and trace to some gravel. The results of grain size distribution tests completed on three selected samples of the sandy silt to sand and silt till are shown on Figure B3 in Appendix B.

The natural water content measured on four samples of the sand and silt till deposit ranges from about 7 to 8 per cent.

The measured SPT "N" values within this deposit range from 14 blows per 0.3 m of penetration to between 100 blows per 0.13 m of penetration and 100 blows per 0.07 m of penetration, indicating a compact to very dense (but typically very dense) relative density.

4.3 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets in Appendix A of this report. The water level in the open boreholes was measured at 4.9 m and 4.7 m below ground surface (corresponding to Elevation 106.8 m and 107.0 m) upon completion of drilling in Boreholes HR-1 and HR-2, respectively.

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring season and periods of precipitation. Given the presence of a deposit of granular fill overlying very stiff to hard/very dense till, perched groundwater conditions can be expected to be present directly above the till deposit.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact for Golder and Principal, conducted an independent review and quality control audit of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
HIGHWAY 401/HOLT ROAD INTERCHANGE STRUCTURE
CLARINGTON, ONTARIO
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing Highway 401/Holt Road Interchange Underpass structure and associated approach embankments. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

As part of the future widening of Highway 401 from Courtice Road easterly to the Regional Municipality of Durham east boundary, and plans to upgrade the Highway 401/Holt Road Interchange, we understand that the preliminary design includes removal of the existing Hwy 401/Holt Road Interchange/Underpass structure and associated ramps, and construction of a new Hwy 401/Holt Road Interchange including a new Underpass structure. Based on the preliminary design completed to date, it is understood that Holt Road will be realigned near the intersection with Hwy 401 and a replacement Underpass structure is to be constructed about 30 m to the east of the existing structure.

The existing four-span structure is about 60 m long with inner span lengths of about 18 m and outside span lengths of about 12 m, and the bridge deck is about 10 m wide. The existing abutments are supported on piles driven into the very dense native till deposits and the piers are shown to be supported on spread footings founded on the native glacial till soil deposits between about Elevation 109.4 m and 108.2 m. The existing approach embankments are up to about 6.5 m high and side slopes are oriented at approximately 2H:1V. Based on visual observations during the current site investigation, the existing bridge foundations appear to have performed satisfactorily to date (i.e. no signs of cracking/settlement) and the approach embankments appear to be stable.

Based on the Draft General Arrangement drawings provided by URS (Holt Road – Conceptual Preferred Plan, on September 13, 2012), the replacement Underpass structure is two spans and will be about 75 m long. The finished pavement grade for Highway 401 is proposed to be maintained at approximately Elevation 112 m and pavement grade for the new realigned Holt Road will be approximately Elevation 120 m, resulting in new approach embankments up to 8 m high.

Based on the proposed Underpass geometry and the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments and piers for the new Holt Road Underpass structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs and risks is provided in Table 1 following the text of this report.

- **Strip or spread footings founded on the very dense sandy silt to sand and silt till:** Strip or spread footings are feasible for support of the new abutments, associated wing walls/retaining walls, and piers at this site, although this foundation type would not permit the use of integral abutments. Temporary protection systems may be required along the outside and median edges of the Highway 401 westbound and eastbound lanes, to facilitate excavation for the construction of the new footings.

- **Footings "perched" on a compacted granular pad in the approach embankment:** "Perched" footings are feasible for support of the new abutments (but not at the piers) and could reduce the need for temporary protection systems along the outside edges of the Highway 401 westbound and eastbound lanes associated with the new abutment construction.

- **Driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and piers, and would permit design of conventional abutments, semi-integral abutments (for tube piles) or integral abutments (for H-piles). It is assumed that the abutment pile caps would be "perched" within the Holt Road approach embankment. Due to the relatively shallow depth to very dense till, pre-augering into the "100-blow" soil is expected to be required, with the piles driven into pre-augered holes. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense till soils.

- **Caissons:** Caissons are feasible for the support of the abutments and piers but precludes the use of integral abutments. This option will be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required. It is assumed that the abutment pile caps would be "perched" within the Holt Road approach embankment, or caissons extended above ground (without caps) to form the bridge piers.

Based on the above considerations, both shallow and deep foundation options are considered feasible and appropriate for the support of the new foundation elements, however from a foundations perspective, shallow foundations are preferable for the support of the new abutments and piers as appropriate foundation conditions (geotechnical resistances) are present at relatively shallow depth on the "100-blow" soil stratum.

6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, piers, and any associated concrete wing walls/retaining walls, strip or spread footings should be founded below any fill or softened/loosened surficial soils, on the very dense sandy silt to sand and silt till deposit. The founding elevation should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Frost Penetration Depths for Southern Ontario)*.

The following maximum (highest) founding elevations are recommended for preliminary design of footings founded on very dense silty sand to sand and silt till.

Foundation Element	Maximum (Highest) Founding Elevation (m)	Approximate Excavation Depth Below Existing Grade (m)
North abutment	107.9	3.8
Piers	108.4*	3.6*
South abutment	108.8	2.9

*Interpolated between boreholes, Draft GA Drawing and DOT BH1.

During detail design, consideration could be given to founding spread/strip footings at higher elevations on the very stiff to hard clayey silt till if consistent subsoil conditions are encountered; however geotechnical resistance values will decrease accordingly.

The footing subgrade should be inspected by the Quality Verification Engineer following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to check that all existing fill, softened/loosened soils or other unsuitable material have been removed. The founding soils will be susceptible to disturbance, and a concrete working slab should be placed on the prepared subgrade as described in Section 6.8.4.

Alternatively, the abutment foundations could be "perched" on a compacted granular pad in the approach embankments above the Highway 401 grade. In this case, the compacted granular pad should have a minimum thickness of 2 m, such that the pad extends below any existing fill and/or loose soil to found on the compact to very dense/very stiff to hard glacial till deposit, encountered at Elevation 109.4 m and 109.9 m at the north and south abutments, respectively. The pad should consist of Special Provision (SP110S13 (aggregates)) Granular 'A' material extending at least 1 m beyond the front and back edge of the abutment footings, then outward and downward at 1H:1V. The granular fill should be placed in accordance with MTO's Special Provision 105S21 and OPSS 501 (Compacting).

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the native very dense sandy silt to sand and silt till or perched on a compacted Granular 'A' pad within the approach embankments founded at or below the preliminary design elevations given in the preceding section, should be designed based on the factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS for 25 mm of settlement) given below.

Founding Stratum	Assumed Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS*
Abutments or pier footings on very dense sandy silt to sand and silt till	3 m	600 kPa	400 kPa
Abutments perched in approach embankments on compacted Granular 'A' pad	3 m	700 kPa	350 kPa

* For 25 mm of settlement

The geotechnical resistances should be reviewed if the selected footing width or founding elevations differ from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary.*

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment and pier locations. During detail design, consideration could be given to founding spread/strip footings at higher elevations on the very stiff to hard clayey silt till if consistent subsoil conditions are encountered; however geotechnical resistance values will decrease accordingly.

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on a concrete working slab that is placed on top of the very dense sandy silt to sand and silt till or on SP110S13 Granular 'A' (Aggregates), the unfactored coefficient of friction, tan δ or tan ϕ', can be taken as follows for preliminary design:

- Cast-in-place concrete footing or concrete working slab: tan δ = 0.67
- Cast-in-place footing or concrete working slab on Granular 'A' or sand and silt till: tan ϕ' = 0.60

6.4 Steel H-Pile or Steel Pipe (Tube) Foundations

6.4.1 Founding Elevations

The abutments, piers and any associated wing walls may be supported on steel H-piles or steel pipe (tube) piles founded in the very dense sandy silt to sand and silt till (having SPT "N" values of greater than 100 blows per 0.3 m of penetration).

The surface of the "100-blow" soils was encountered at approximately Elevation 107.9 m to 108.8 m at the north and south abutments, respectively. The following pile tip elevations may be used for preliminary design purposes, assuming piles are driven at least 1 m into the "100-blow" material:

Foundation Element	Estimated Design Pile Tip Elevation
North abutment	106.5 m
Centre Pier*	107.0 m
South abutment	107.5 m

*Interpolated between boreholes and DOT BH 1

If integral or semi-integral abutments are preferred, pre-auguring into the very dense till soils may be required to reduce the potential for driving the piles out of alignment, installing CSP's, or damaging the pile tips in the very dense till deposit to achieve a minimum 5 m pile length (typically required for integral abutments). If piles are adopted for support of the centre pier, pre-augering into the “100-blow” soil may be required to achieve a minimum recommended pile length of 3 m below the pile cap. The foundation (pile tip) elevations will have to be confirmed during detail design once details of the proposed pile cap elevations are available.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the till deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations).* In very dense and/or bouldery soils, as may be encountered at this site, driving shoes (such as Titus Standard “H” Bearing Pile Points) are preferred over flange plates (OPSD 3000.100). If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

The pile caps for the new abutments should be provided with a minimum of 1.2 m soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101).

6.4.2 Axial Geotechnical Resistance/Reaction

For HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored geotechnical axial resistance at ULS may be taken as 1,600 kN, and the geotechnical axial reaction at SLS (for 25 mm of settlement) may be taken as 1,400 kN. Similar axial resistances may be used in the design for closed-end, concrete filled 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 6.4 mm (1/4 in.).

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO Standard Structural Drawing SS103-11) during the final stages of driving to achieve the appropriate ultimate capacity. An appropriate pile driving note should be included in the foundation drawing, as per MTO’s Structural Manual (2008) Section 3.3.3.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements.

6.4.3 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by the horizontal component of battered piles. The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equation given below (CFEM, 1992, as noted in Section C6.8.7.1 (Table C6.5) and in Section C6.8.7.3 of the *Commentary to CHBDC, 2006*) for the cohesionless soils at this site:

$$k_h = \frac{n_h z}{B}$$ where k_h is the coefficient of horizontal subgrade reaction (kPa/m);

n_h is the constant of subgrade reaction (kPa/m);

z is the depth (m); and

B is the pile diameter / width (m).

The following values of n_h may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the profile on Drawing 1:

Soil Unit	n_h (kPa/m)
Embankment fill (assuming engineered earth fill)	5,000
Loose sand within CSP (if applicable)	2,200
Very stiff clayey silt with sand to compact sandy silt till	7,000
Very dense sandy silt to sand and silt till	18,000

A maximum factored lateral resistance of 120 kN at ULS, and a maximum lateral resistance of 50 kN at SLS (for 10 mm of horizontal deflection at pile cap level) is recommended for HP 310x110 piles at this preliminary stage; however the values should be checked and modified as necessary at the detail design stage. These values are based on the “Assessed Horizontal Passive Resistance” (provided in Table C6.4 of the *Commentary* to the *CHBDC)*, and Geotechnical Reaction at SLS interpreted for the site conditions and pile size presented above.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.5 Caisson Foundations

6.5.1 Founding Elevations

The abutments, piers and any associated wing walls may be supported on caissons founded within the very dense sandy silt to sand and silt till (having SPT "N" values of greater than 100 blows per 0.3 m of penetration). The surface of the "100-blow" soils was encountered at approximately Elevation 107.9 m to 108.8 m at the north and south abutments, respectively. The following caisson founding elevations may be used for preliminary design purposes, assuming a minimum 2.0 m long socket into the "100-blow" till deposit:

Foundation Element	Estimated Design Caisson Founding Elevation
North abutment	105.5 m
Piers	106.0 m
South abutment	106.5 m

The surficial soils consist of granular fill which may contain perched groundwater above the till deposit. Temporary liners may be required to support the granular fill soils and saturated cohesionless till soils during construction, especially if perched water conditions are present. The performance of caissons will depend on the final cleaning and verification of the subgrade quality (very dense sandy silt to sand and silt till) at the base of the caissons. The Ontario Occupational Health and Safety Act (2012) outlines appropriate safety procedures and requirements that must be implemented prior to entry of personnel into the caissons for inspection of the base or alternatively, the inspections may be carried out remotely using visual recording equipment.

The caisson caps for the new foundations should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101) unless the caps are positioned at the top of the columns.

6.5.2 Axial Geotechnical Resistance/Reaction

The caissons will derive the majority of their capacity from base resistance, although some shaft friction has also been taken into account based on "socketing" approximately 2.0 m into the "100-blow" till deposit. Using the preliminary design elevations given above, and assuming a 1.2 m diameter caisson, the factored geotechnical resistance axial at ULS may be taken as 5,600 kN and the geotechnical axial reaction at SLS (for 25 mm of settlement) may be taken as 4,500 kN for preliminary design purposes. These values assume the caisson base is properly cleaned and inspected.

These preliminary geotechnical resistances will have to be re-evaluated and modified as necessary during detail design in consideration of any additional subsurface investigation and when more design details are available.

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 6.4.3.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and on any wing walls/retaining 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 the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading may also need to be taken into account in the design.

6.6.1 Static Considerations

The following recommendations are made concerning the design of the abutment walls and any associated wing walls or retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls.

- Select, free-draining granular fill meeting the specifications of SP110S13 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II (but with less than 5 percent passing the 200 sieve) should be used as backfill behind the walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (Compacting). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to such sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirements).

- A minimum compaction surcharge of 12 kPa should be included for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with SP105S10 *(Compacting).* Other surcharge loadings should be accounted for in the design as required.

- The granular fill may be placed either in a zone with the width equal to at least 1.2 m behind the back of the walls (for a restrained wall see Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (for an unrestrained wall see Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For a restrained wall, the pressures are based on any existing and new approach embankment fill materials and the following parameters (unfactored) may be used:

	Earth Fill
Soil unit weight:	21 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For an unrestrained wall, where the pressures are based on SP110S13 (Aggregates) Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

	Granular A	Granular B Type II
Soil unit weight	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure Active, K_a At rest, K_o	 0.27 0.43	 0.27 0.43

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary to the CHBDC*.

6.6.2 Seismic Considerations

Seismic loading may also need to be considered in accordance with Section 4.6.4 of *CHBDC (2006)*, as such loading can result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining walls.

According to Table C4.2 of the *Commentary to the CHBDC*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Durham area is 0.05. The site-specific peak ground acceleration (PGA) is 0.027g based on the NRC website; however, the more conservative *CHBDC* value has been used in the assessment. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of *CHBDC* (2006). Based on experience, for the subsurface conditions at this site, a 20 per cent amplification of the ground motion may occur, resulting in an increase in the peak horizontal ground surface acceleration (PGA) from 0.05g to approximately 0.06g. In accordance with Section 4.4.5.1 of CHBDC (2006) and the MTO Bridge Office Policy Memo *“Clarification of What is Considered a Lifeline, Emergency or Other Bridge for Seismic Design Ontario”* (MTO, 2011), seismic analysis is not required for structures located in seismic Performance Zone 1 that are not classified as “lifeline” structures.

6.7 Approach Embankments

The new Highway 401/Holt Road Underpass structure will require placement of engineered fill for the construction of the approach embankments. The existing ground surface at the north and south abutments is at about Elevation 111 m and 113 m, respectively, and the proposed realigned Holt Road grade is at about Elevation 119 m at the abutment locations, resulting in embankments up to about 8 m high.

In accordance with MTO’s standard practice, a minimum 2 m wide bench should be provided where embankment slopes are equal to or greater than 8 m high. Therefore, for preliminary design it is suggested that 2 m wide mid-height benches be incorporated into the approach embankments near the abutments to reduce the uninterrupted slope length to less than 8 m. To reduce erosion of the slopes due to surface water runoff, placement of topsoil and seeding (OPSS 572) is recommended as soon as practicable after construction of the embankments. Consideration may also be given to the use of armoured drainage channels to direct surface water flow from the Holt Road grade to the Highway 401 grade, if applicable.

6.7.1 Subgrade Preparation and Embankment Construction

It is not known what the existing Underpass approach embankment materials consist of; however, the new approach embankments will likely be constructed while the existing underpass remains in use. Therefor, the existing approach embankment material will likely not be available for re-use in the new approach embankments but could be used elsewhere on site where staging permits.

Prior to placing any embankment fill, all topsoil, organic mater and existing loose fill should be stripped from below the approach embankment areas. Considering loose fill and fill containing organics was encountered in both Borehole HR-1 and HR-2, it is recommended that all existing fills be removed from within the approach embankment footprint where fill heights are in excess of 4.5 m. If existing fills are to remain below the embankments/ramps where fills are less than 4.5 m there should be a transition zone to avoid abrupt differential settlements that could be propagated to the road surface.

Any new embankment fill should be placed and compacted in accordance with SP 206S03 (Earth Excavation and Grading), and SP105S10 (Compacting), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

The use of suitable granular fill for the approach embankments is recommended rather than the use of cohesive fill, since the majority of settlement of granular fills would occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction (refer to Section 6.7.3).

6.7.2 Embankment Stability

Preliminary static and seismic slope stability analyses have been performed for the Holt Road approach embankments, using the commercially available program *Slide (version 6.017),* produced by Rocscience Inc., to check that the target minimum factor of safety is achieved.

Static Stability Analysis

A target minimum factor of safety of 1.3 is normally adopted in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed embankment construction on this project, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analysis, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
New embankment fill (granular fill)	21	32°	-
Existing fill	21	30°	-
Compact sandy silt/very stiff clayey silt with sand till	19	32°	-
Very Dense Sand and Silt Till	20	35°	-

A groundwater level at Elevation 107 m (as measured in the current boreholes HR-1 and HR-2) was modelled in the analysis.

The stability analyses were completed for an overall 8 m high slope using the parameters outlined above and assume that all existing fills (containing organics and rootlets) are completely stripped from below the approach embankment footprint prior to placing the new embankment fill. The results of the static global stability analysis indicate that a minimum factor of safety greater than 1.3 is achieved for 8 m high slopes oriented no steeper than 2H:1V. The result of the analysis at the south approach embankment is shown on Figure 1. This preliminary assessment of the slope stability of the approach embankments should be reviewed and confirmed during detail design based on the refined geometry and additional subsurface information.

Short-term shallow sloughing (i.e. surficial failures) could occur on the 2H:1V slope faces, which could be mitigated in the long-term by providing well-vegetated slopes.

Seismic Stability Analysis

Under seismic conditions, the stability of the embankment slopes is assessed using conventional pseudo-static methods of slope stability analysis under the earthquake-induced peak ground acceleration. A calculated factor of safety of 1.0 is considered appropriate for global stability under seismic conditions. A preliminary seismic global stability analysis has been performed for the new embankment slopes, using the parameters summarized above.

The preliminary pseudo-static seismic slope stability analyses for a 2H:1V slope configuration indicate that the embankment slopes will have a factor of safety of equal to 1.3 against deep-seated slope instability, using a peak ground acceleration of 0.06g. The result of the pseudo-static stability analysis at the south approach embankment is shown on Figure 2.

6.7.3 Approach Embankment Settlement

Settlement analysis for the anticipated foundation soil conditions below the new approach embankments was carried out using the commercially available computer program *Settle-3D* (version 2.015), produced by Rocscience, using estimated elastic deformation moduli as given below, based on correlations with the SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Conditions	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
New embankment fill (granular fill)	21	Not considered
Compact sandy silt to very stiff clayey silt till	19	25 MPa
Very dense sandy silt to sand and silt till	20	100 MPa

The settlement analysis assumes any existing fills (i.e. containing organics and rootlets) are completely stripped and the new embankment fill is placed and compacted above the relatively undisturbed native soils.

Based on this assessment, the settlement of the foundation soils under the new up to 8 m high approach embankments is estimated to be less than 25 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed when profiles and grades are determined during the Detail Design, with particular emphasis on the thickness and properties of any surficial soil deposits within the embankment footprint.

The above estimates do not include compression of the new embankment fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression typically ranges from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.8 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design and/or require non-standard special provisions during construction.

6.8.1 Open Cut Excavation

The temporary excavations for spread/strip footings would extend to a depth of up to 3.8 m below existing grade through the existing loose to compact fill and compact sandy silt to very stiff clayey silt till deposits, and to the very dense sandy silt to sand and silt till.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and compact/very stiff portion of the surficial deposits are classified as Type 3 soil, while the lower very dense till deposits are classified as a Type 2 material, according to OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

6.8.2 Temporary Protection Systems

It is anticipated that temporary protection systems may be required along the outside and median edges of the Highway 401 westbound and eastbound lanes, to facilitate the construction of new footings and/or pile caps. These temporary protection systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided any adjacent utilities can tolerate this magnitude of deformation.

6.8.3 Groundwater Control During Construction

Excavations for construction of the new abutment and pier foundations are not expected to extend below the groundwater level at the site, which has been measured at about Elevation 107 m in the vicinity of the abutments. Groundwater seepage should be anticipated from the native till deposits (including cohesionless lenses or interlayers within the till) or overlying fill soils as will be encountered in all of the foundation excavations and perched water may be present within the cohesionless fill deposits above the till, however it is expected that such seepage volumes will be minor and could be controlled by pumping from properly filtered sumps within the foundation excavations. It is anticipated that a Permit to Take Water (PTTW) would not be required for control of the groundwater seepage at this site, but this requirement should be determined at Detail Design.

As discussed in Section 6.5, running or flowing water-bearing cohesionless soil strata could be encountered during or after drilling of caissons. If caisson foundations are adopted, temporary or permanent caisson liners would be required to support the soils during construction, and special methods such as the use of drilling mud and placement of concrete by tremie methods may be required to keep the hole open and minimize disturbance to the caisson base.

6.8.4 Subgrade Protection

The sandy silt to sand and silt till (and any interlayers, if present) that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a 100 mm thick concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. An NSSP should be provided at the detail design stage.

6.8.5 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further assessment is recommended in any future investigation at this site, to delineate the presence of cobbles and boulders (if possible) to aid the contractor to assess the impact on foundation construction.

6.9 Recommendations for Further Work During Detail Design

Additional boreholes are recommended at each of the foundation elements during the Detail Design stage, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- Abutments and piers:
  - Determine whether foundations can be founded at higher elevations (in the very stiff cohesive till) thereby reducing subexcavation depths.
  - Confirm the surface of the “100-blow” stratum across each foundation element to confirm the bearing resistance and founding elevation for shallow foundations and to confirm the tip or base elevation for deep piles.
  - Confirmation of the stabilized groundwater elevation across the site and potential dewatering requirements.
  - Observation of the presence of cobbles and/or boulders within the soil deposits to assess the potential for impact on the installation of deep foundations and/or protection systems.
- Approach Embankments:
  - Determine thickness, extent and characteristics of existing fill materials and explore the option of constructing new embankments over existing fills.

- Assessment/confirmation of the subsurface stratigraphy and foundation soil properties for approach embankment settlement and stability analysis.
- Confirmation of the stabilized groundwater elevation across the site and identify the presence of any perched water.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Kevin Bentley, P.Eng., a geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact, and principal with Golder, conducted an independent review and quality control audit of this report.

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Rocscience Inc., (2012) *Slide* (Version 6.017) [Computer Program]

ASTM International

ASTM D1556 Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils

Contract Design and Estimating and Documentation (CDED)

SP110S13 Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

SP105S10 Construction Specification for Compacting

SP105S21 Amendment to OPSS 501

Ontario Provincial Standard Specifications (OPSS)

OPSS 501 Construction Specification for Compacting

OPSS 539 Construction Specification for Temporary Protection Systems

OPSS 572 Construction Specification for Seed and Cover

OPSS 902 Construction Specification for Excavating and Backfilling Structures

OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3001.100	Foundation Piles - Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

TABLE 1 – COMPARISON OF PRELIMINARY FOUNDATION ALTERNATIVES

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Spread/strip footings on very dense sandy silt to sand and silt till / hard clayey silt till	• Appropriate geotechnical axial resistances readily available for support of piers, abutments and associated wing walls/retaining walls • Adjacent existing structure piers supported on shallow foundations, and appears to have performed satisfactory • Standard construction operation	• Temporary excavations (to a depth of up to 3.0 m below the existing grade) may require temporary excavation support • Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower, but adequate, geotechnical resistances available than for deep foundations	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration	• Traffic disruption to Hwy 401 can be reduced if temporary protection systems are used • Relatively low risk of significant groundwater seepage for excavations to depths up to 2.5 m below ground surface • Founding depths to competent till may be deeper than 2.5 m below ground surface at some foundation locations depending on results of detail design investigation
Spread/strip footings perched on compacted granular pad in approach embankment fill (abutments only)	• Feasible for support of abutments and associated wing walls/retaining walls • Abutment footings can be maintained higher than footings founded on till deposit and do not require subexcavation or temporary protection systems	• Precludes use of integral abutments; potentially greater maintenance required at abutments • Lower geotechnical resistances compared to shallow foundations on native till soils • Existing fill (up to 2.3 m thick) will likely need to be stripped from below approach embankment footprint	• Less expensive than deep foundations, although bridge maintenance costs may be higher due to non-integral abutment configuration	• Geotechnical resistance relies on quality of placement and compaction of engineered fill • Potential for differential settlements if existing fill is not stripped from below approach embankments and due to different foundation strata at piers compared to abutments

TABLE 1 – COMPARISON OF PRELIMINARY FOUNDATION ALTERNATIVES

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Steel H-piles or tube piles driven to found within the very dense sandy silt to sand and silt till deposit	• Subsurface conditions are appropriate for support of piers, abutments and associated wing walls/retaining walls • Limited temporary excavation for pile caps compared to deeper excavation and temporary excavation support requirements for shallow footings • Allows for integral abutment construction (steel H-piles) • Higher geotechnical axial resistance available compared to shallow foundations	• Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles “hanging up” and not achieving a minimum pile embedment length (typically 5 m) for integral abutment design • If piles “hang up”, pre-augering may be required • Potential for traffic disruption due to requirement for large piling equipment • Tube piles not normally accepted by MTO for integral abutment design	• Lower relative cost compared with caisson option • Higher relative cost compared to shallow foundation options • Steel H-piles typically lower cost than tube piles	• Conventional construction methods for H-pile foundations • Potential for piles to “hang up” on cobbles/boulders and pre-augering may be required
Caissons founded within the very dense sandy silt to sand and silt till	• Subsurface conditions are appropriate for support of piers, abutments and wing walls/ retaining walls • Limited temporary excavation for pile caps compared to deeper excavation and temporary excavation support requirements for shallow footings • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles • Caissons can be designed to be continuous to act as columns above ground, thereby eliminating caisson/pile caps below grade and	• Temporary or permanent liners may be required through loose to compact granular fills • Precludes use of integral abutments • Large staging area required and will likely lead to traffic disruption	• Higher cost compared with shallow foundations or steel H-piles/tube piles	• Risk of loosening soils at base of caissons and potential need for temporary or permanent lines if water table is higher than expected • Difficulties augering through till soil if cobbles/boulders present as should be anticipated in the glacial till

TABLE 1 – COMPARISON OF PRELIMINARY FOUNDATION ALTERNATIVES

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
	associated subexcavation requirements			

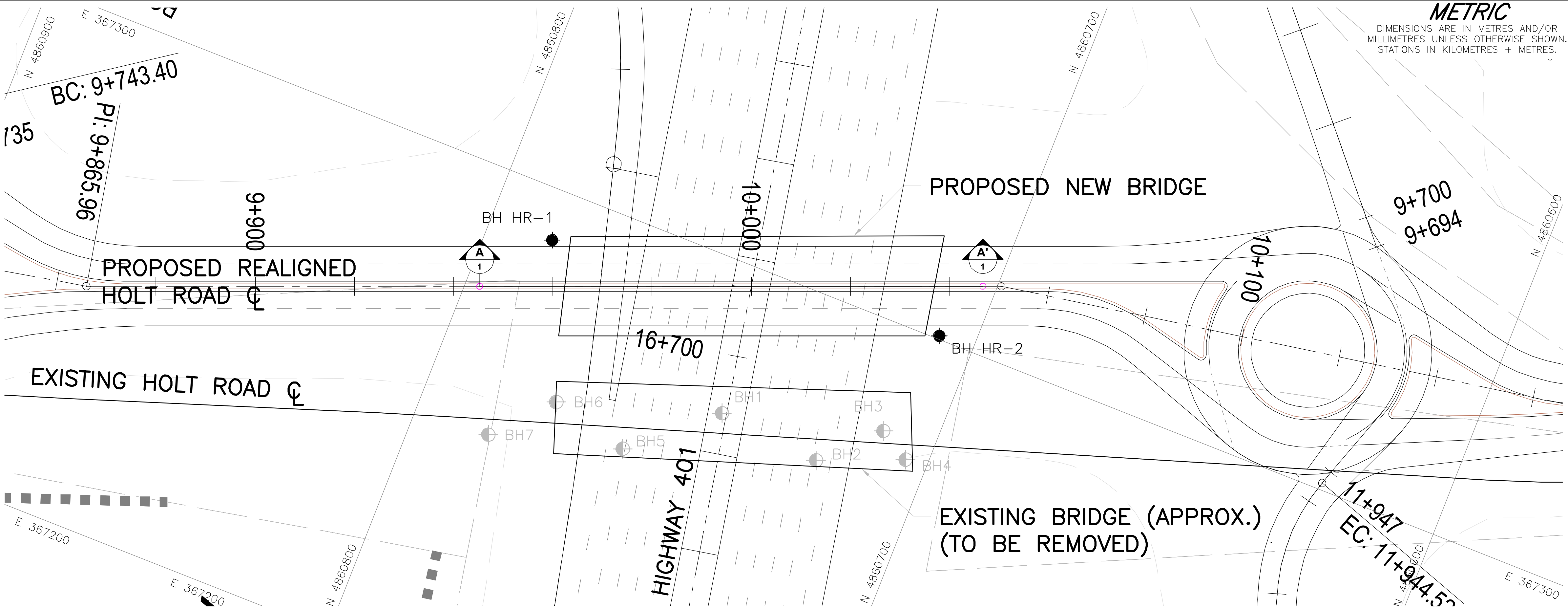
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2101-08-00

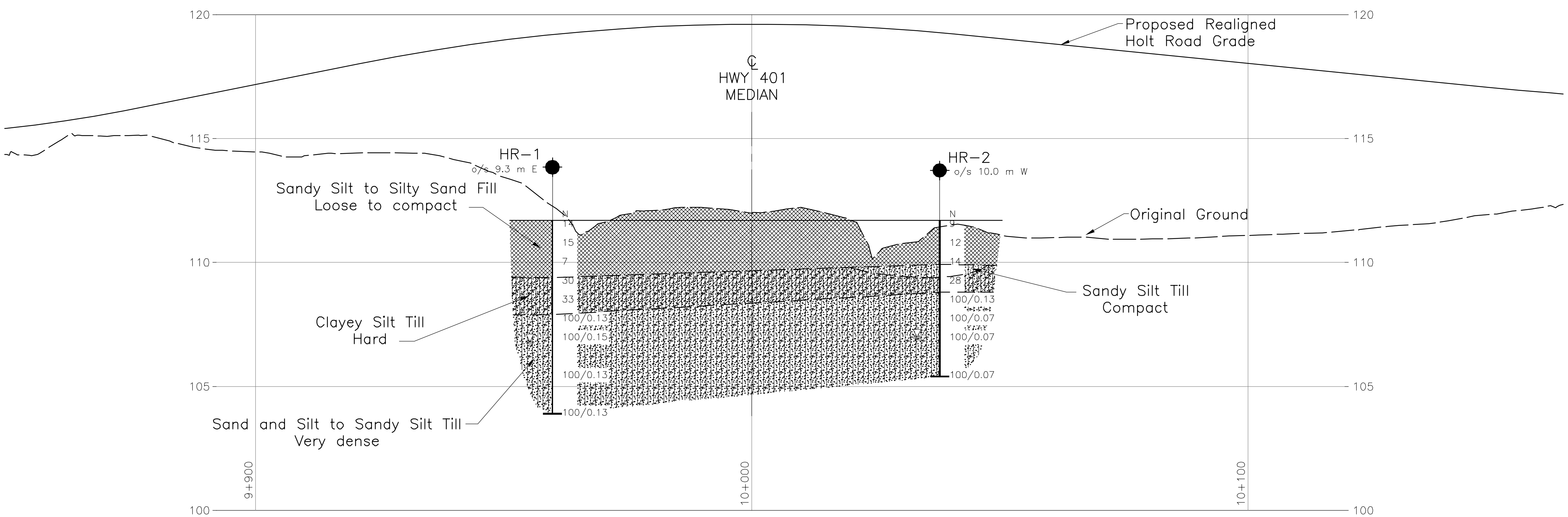
SHEET

HIGHWAY 401
HOLT ROAD INTERCHANGE STRUCTURE
BOREHOLE LOCATIONS AND SOIL STRATA

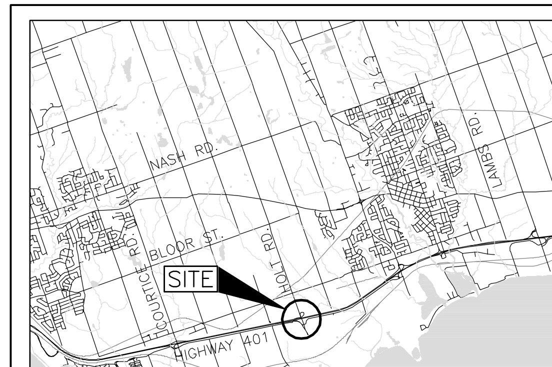
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
10 0 10 20 m



A-A' 1 CENTRELINE PROFILE ALONG REALIGNED HOLT ROAD
HORIZONTAL SCALE
10 0 10 20 m
2 0 2 4 m
VERTICAL SCALE



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (1961)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
BH HR-1	111.7	4860786.9	367290.0
BH HR-2	111.7	4860707.2	367300.7
BH1	112.8	4860742.3	367270.1
BH2	112.5	4860721.2	367268.3
BH3	111.9	4860710.7	367278.7
BH4	111.9	4860704.4	367275.0
BH5	112.9	4860758.3	367256.1
BH6	111.6	4860774.2	367260.0
BH7	111.5	4860784.5	367248.8

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.

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plan provided in digital format by URS, drawing file no. 120921-X-Design_Holt Preferred_ACAD 2007.dwg, received January 3, 2013

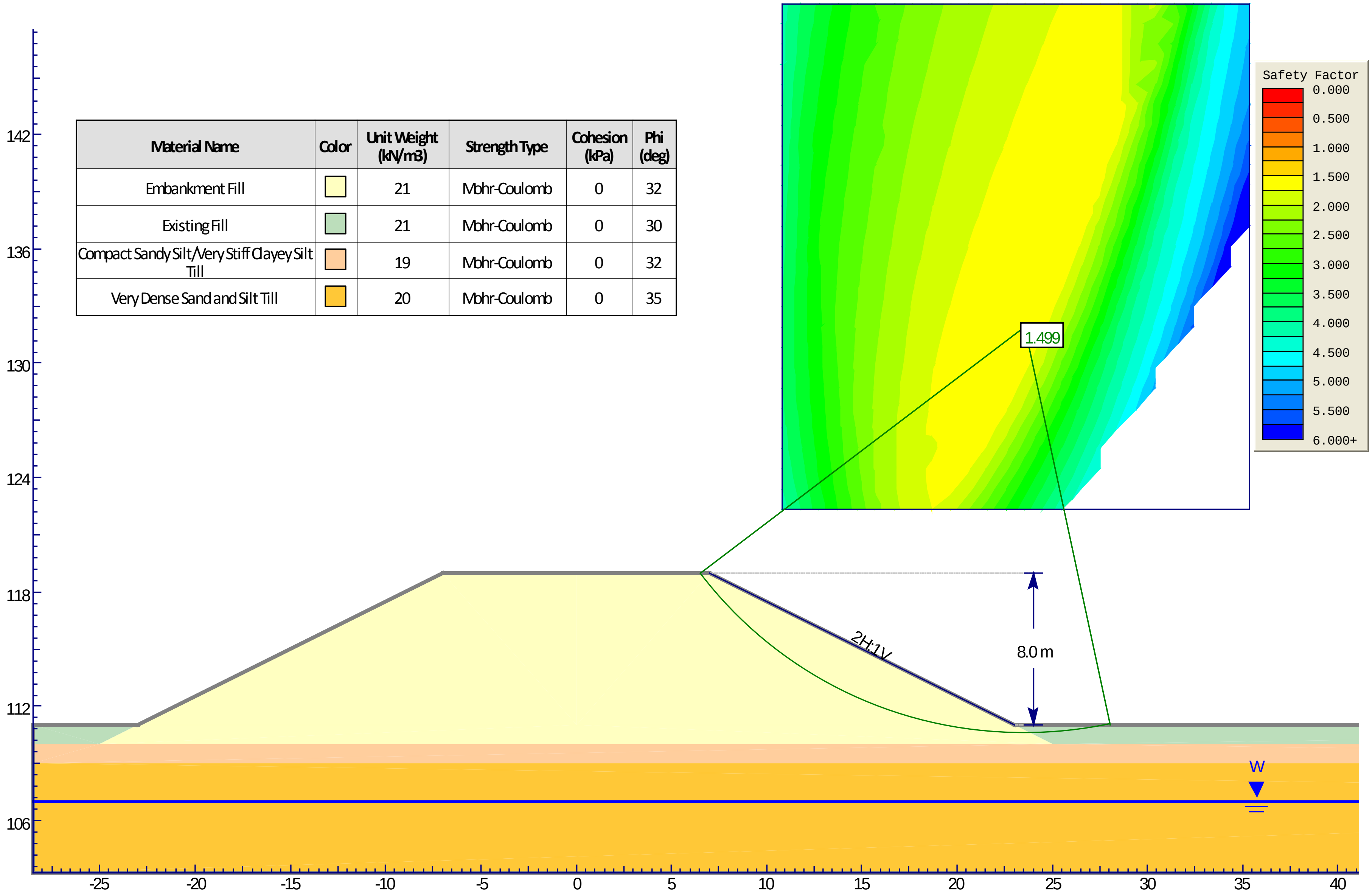
NO.	DATE	BY	REVISION

Geocres No. 30M15-119

HWY. 401		PROJECT NO. 09-1111-0019	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: 26/03/2013	SITE:
DRAWN: JFC	CHKD. KJB	APPD. JMAC	DWG. 1

STATIC GLOBAL STABILITY
HWY 401/HOLT ROAD INTERCHANGE, SOUTH APPROACH EMBANKMENT

Figure 1

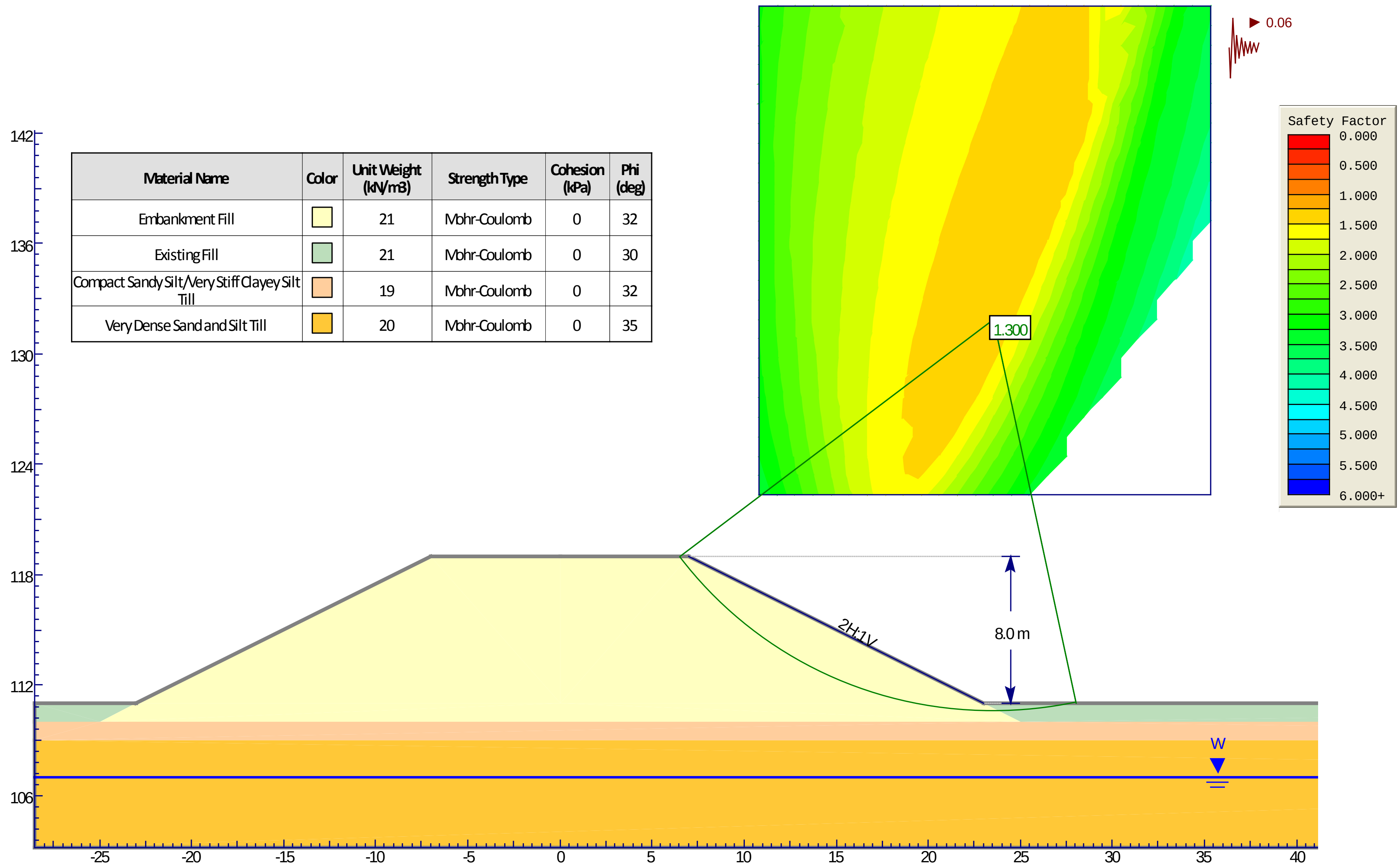


Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	32
Existing Fill		21	Mohr-Coulomb	0	30
Compact Sandy Silt/Very Stiff Clayey Silt Till		19	Mohr-Coulomb	0	32
Very Dense Sand and Silt Till		20	Mohr-Coulomb	0	35

Date: January 2012
Project No: 09-1111-0019

Analysis By: MWK **Reviewed By:** KJB

Golder Associates



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	32
Existing Fill		21	Mohr-Coulomb	0	30
Compact Sandy Silt/Very Stiff Clayey Silt Till		19	Mohr-Coulomb	0	32
Very Dense Sand and Silt Till		20	Mohr-Coulomb	0	35

Date: January 2012
Project No: 09-1111-0019

Analysis By: MWK **Reviewed By:** KJB

APPENDIX A

Borehole Records

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
SS	Split-spoon
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

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.)

Dynamic Cone 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, 60º 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

Piezo-Cone Penetration Test (CPT)
A 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 (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, s_u kPa	c_u, s_u 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 limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
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 (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

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

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
σ_1, σ_2, σ_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 $\gamma = \rho g$ (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 $= (w_l - w_p)$
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_α	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
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c'	effective cohesion
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	$(\sigma_1 - \sigma_3)/2$ 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

RECORD OF BOREHOLE No HR-1

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0019
W.P. 2101-08-00
LOCATION N 4860786.9 ;E 367290.0
ORIGINATED BY BM
DIST HWY 401
BOREHOLE TYPE 108 mm O.D. Continuous Flight Solid Stem Power Augering
COMPILED BY MS
DATUM Geodetic
DATE November 22, 2012
CHECKED BY MWK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
111.7	GROUND SURFACE					
0.0	Sandy silt to silty sand, trace clay, trace to some gravel, containing rootlets and organics (FILL) Loose to compact Brown to grey Moist	1	SS	14		
		2	SS	15		
		3	SS	7		
109.4 / 2.3	CLAYEY SILT with sand, some gravel (TILL) Hard Grey Moist	4	SS	30		12 38 38 12
		5	SS	33		
107.9 / 3.8	SAND and SILT, some clay, trace to some gravel (TILL) Very dense Grey Moist	6	SS	100/0.13		16 40 32 12
		7	SS	100/0.15	∇	
		8	SS	100/0.13		5 40 43 12
104.7 / 7.0	Sandy SILT, trace clay, trace to some gravel (TILL) Very dense Grey Moist					
103.9 / 7.8	END OF BOREHOLE	9	SS	100/0.13		

NOTES:

1. Water level in open borehole measured at a depth of 4.9 m below ground surface (Elev. 106.8 m) on completion of drilling.

Elevation scale: 111, 110, 109, 108, 107, 106, 105, 104

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 (%) 10 20 30

UNIT WEIGHT γ kN/m³

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

Golder Associates

RECORD OF BOREHOLE No HR-2

SHEET 1 OF 1 **METRIC**

PROJECT 09-1111-0019

W.P. 2101-08-00 LOCATION N 4860707.2 ;E 367300.7 ORIGINATED BY BM

DIST HWY 401 BOREHOLE TYPE 108 mm O.D. Continuous Flight Solid Stem Power Augering COMPILED BY MS

DATUM Geodetic DATE November 22, 2012 CHECKED BY MWK

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
111.7	GROUND SURFACE							
0.0	Sandy silt, trace clay, trace gravel, containing rootlets and organics (FILL) Loose to compact Brown to grey Moist	1	SS	9		111		
		2	SS	12			o	
109.9 1.8	Sandy SILT, trace clay, trace gravel (TILL) Compact Grey Moist	3	SS	14		110		
109.4 2.3	CLAYEY SILT, some sand, trace gravel (TILL) Very stiff Grey Moist	4	SS	28		109	Wp–w–WL	1 12 52 35
108.8 2.9	SAND and SILT, some clay, trace to some gravel (TILL) Very dense Grey Moist	5	SS	100/0.13			o	5 47 34 14
		6	SS	100/0.07		108		
		7	SS	100/0.07	∇	107	o	
						106		
105.4		8	SS	100/0.07				
6.3	END OF BOREHOLE							

NOTES:

1. Water level in open borehole measured at a depth of 4.7 m below ground surface (Elev. 107.0 m) on completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O 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 (%) 10 20 30

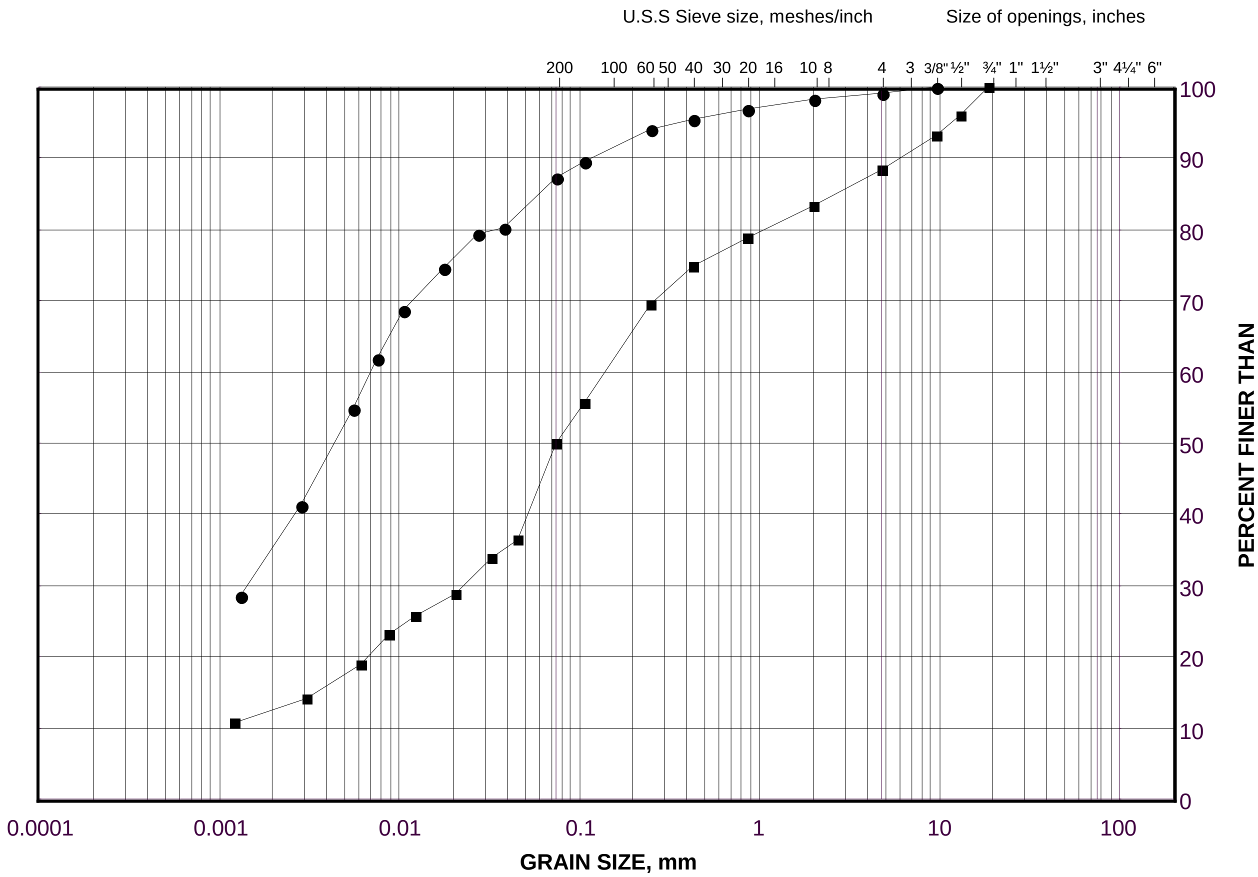
UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS Clayey Silt Till	FIGURE B1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HR-2	4	109.1
■	HR-1	4	109.1

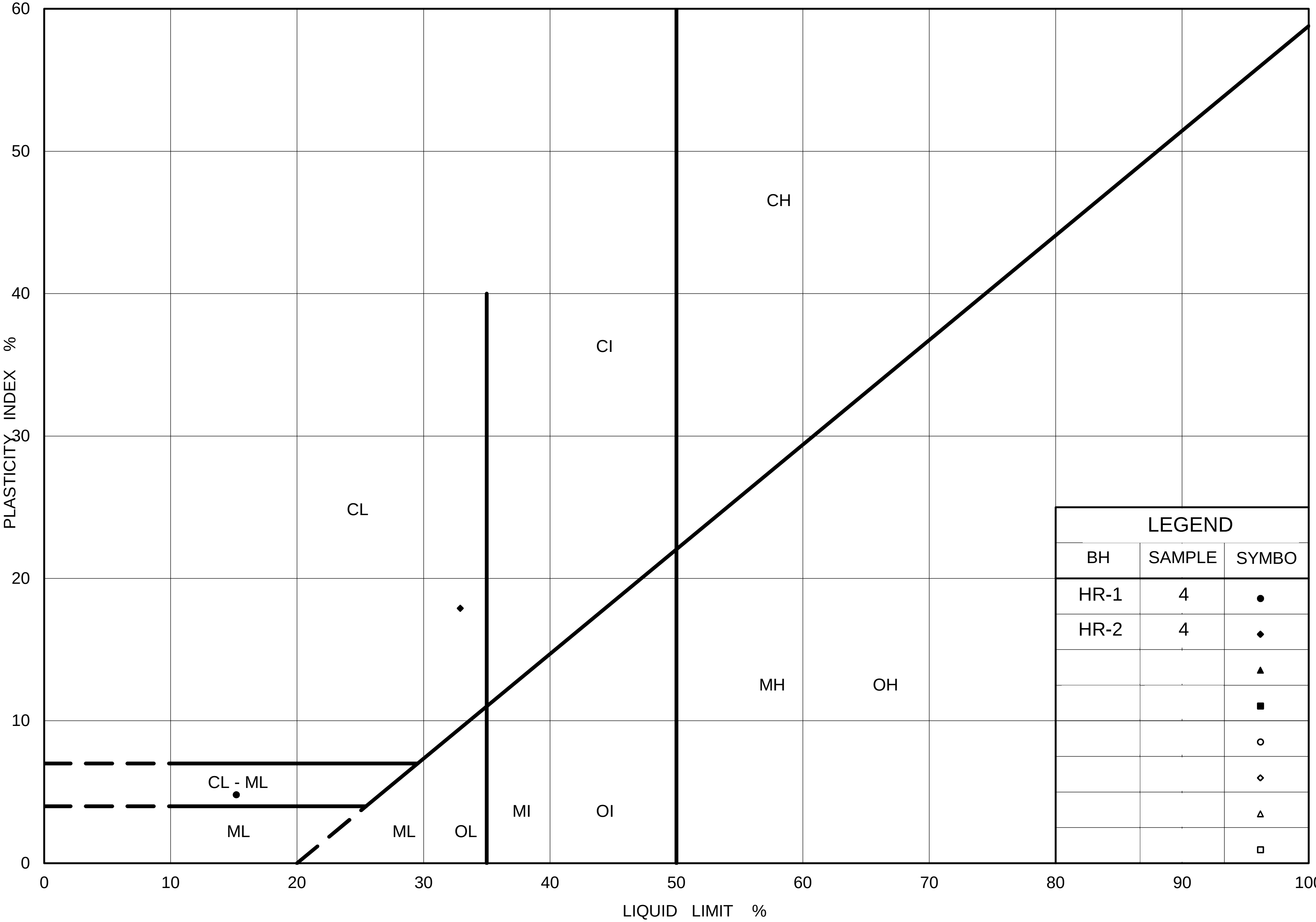
Project Number: 09-1111-0019

Checked By: ______________________

Golder Associates

Date: 24-Jan-13

Oct 75, FF-S-21



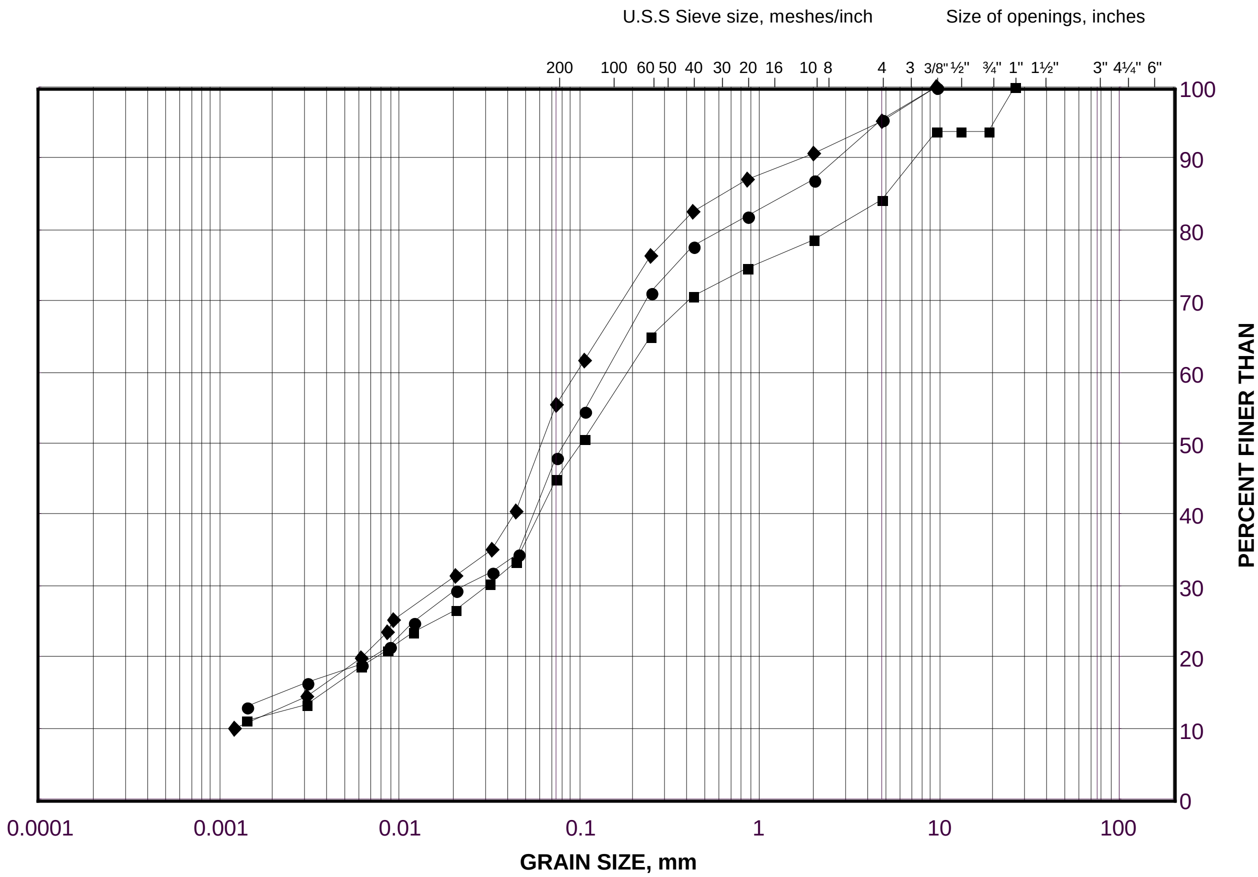
LEGEND		
BH	SAMPLE	SYMBO
HR-1	4	●
HR-2	4	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation Ontario	PLASTICITY CHART Clayey Silt (Till)	Figure No. B2 Project No. 09-1111-0019 Checked By: MWK

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sandy Silt to Sand and Silt Till

FIGURE B3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	HR-2	5	108.6
■	HR-1	6	107.8
◆	HR-1	8	105.5

Project Number: 09-1111-0019

Checked By: ______________________

Golder Associates

Date: 24-Jan-13

APPENDIX C

Borehole Logs – Previous Investigation

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58 BORE HOLE NO. 1

JOB 61-F-15 STATION See drawing

DATUM 370.0' COMPILED BY B.K.

BORING DATE Mar. 2/61 CHECKED BY V.K.

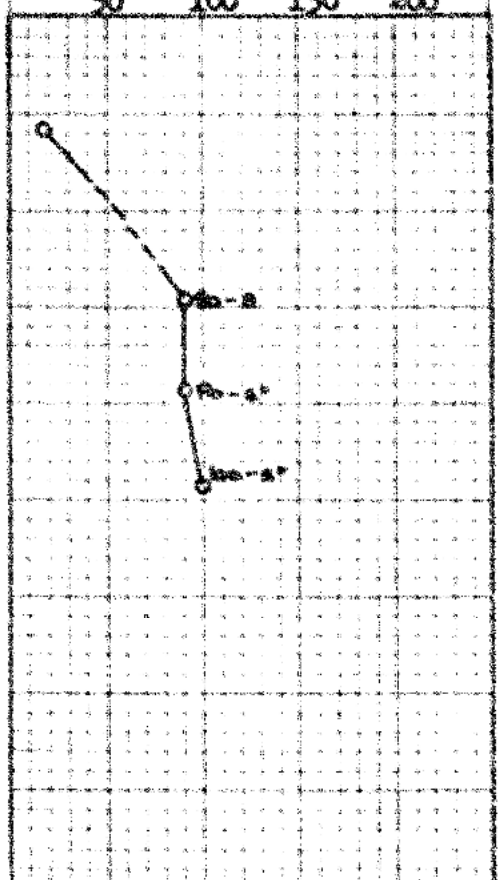
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

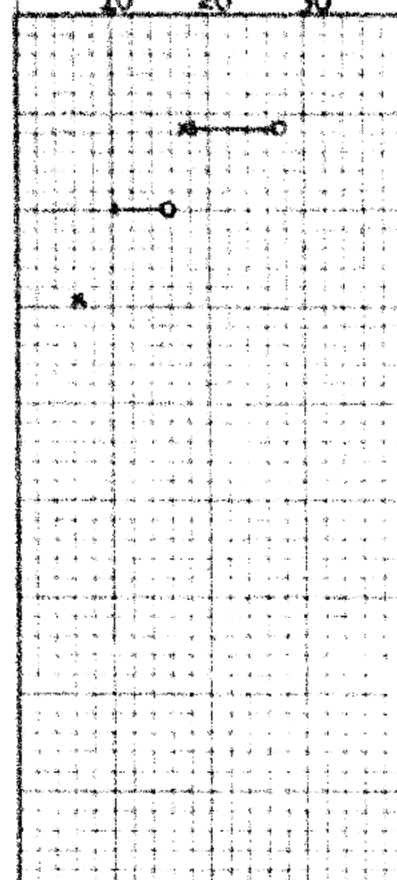
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel	370.0	0
	Granular Fill	365.0	
	Dark brown clayey silt (Topsoil)	363.0	
	Light brown pebbly silty sand (till)	351.0	10
	Grey pebbly silty sand (till)	345.5	20
	End of borehole		30
			40

STRENGTH AND PENETRATION RESISTANCE P.S.F. BLOWS/FT.



CONSISTENCY

MOIST. CONTENT % DRY WT.



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
T2	=
S3	
S4	145.8
S5	-
S6	-

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
JOB 61-F-15
DATUM 369.0'
BORING DATE Mar. 2/61

BORE HOLE NO. 2
STATION See Drawing
COMPILED BY B.K.
CHECKED BY V.K.

2" DIA. SPLIT TUBE
2" SHELB TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

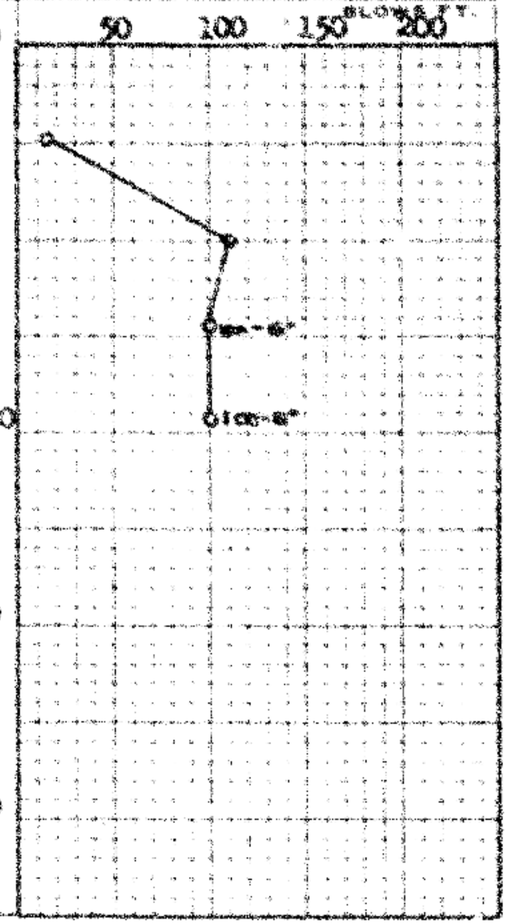
LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	369.0		
	Granular Fill	365.0		
	Dark brown clayey silt (topsoil)	362.0	S1	-
	Light brown pebbly silty sand (Till)	351.0	S2	-
			S3	-
	Grey pebbly silty sand (till)	346.0	S4	-
	End of borehole.			

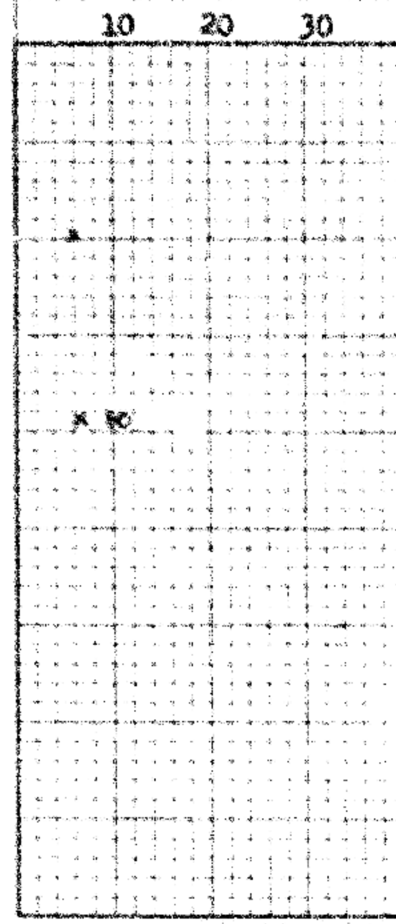
DEPTH FEET: 0, 10, 20, 30, 40

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT % DRY WT.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58 BORE HOLE NO. 3

JOB 61-P-15 STATION See Drawing

DATUM 367.0' COMPILED BY B.K.

BORING DATE Mar. 2/61 CHECKED BY V.K.

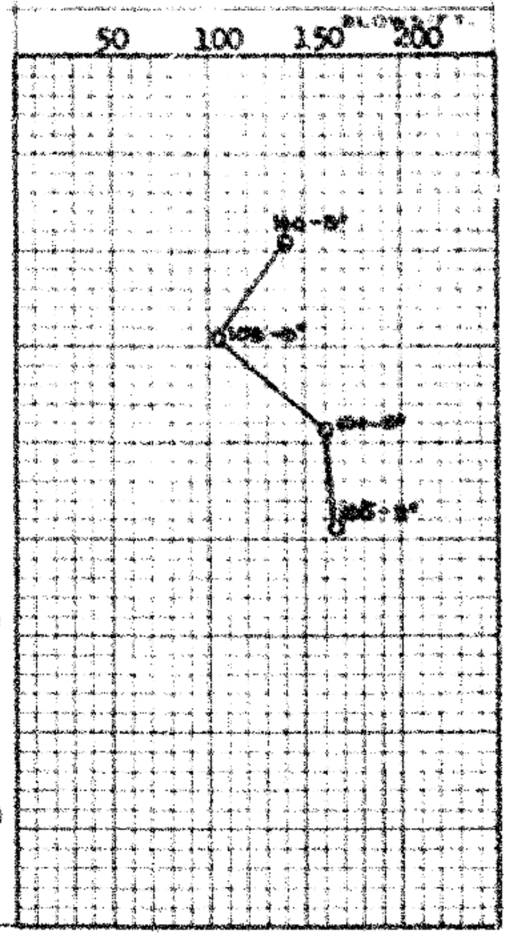
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +S
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

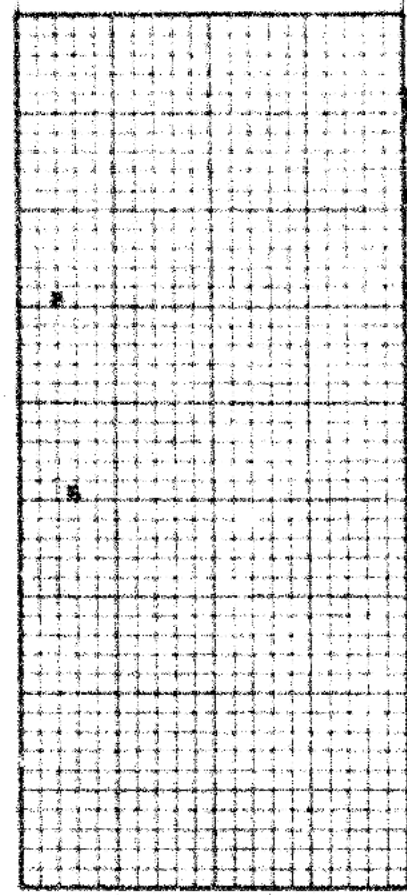
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	367.0	0		
	Granular Fill	366.0			
	Dark brown silty clay (topsoil)	361.5		S1	-
	Light brown pebbly silty sand (till)	351.0	10	S2	-
				S3	-
	Grey pebbly silty sand till	342.0	20	S4	-
				S5	-
	End of borehole		30		
			40		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT - % DRY WT.



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58

JOB 61-F-15

DATUM 367.0'

BORING DATE Mar. 2/61

BORE HOLE NO. 4

STATION See Drawing

COMPILED BY B.K.

CHECKED BY V.K.

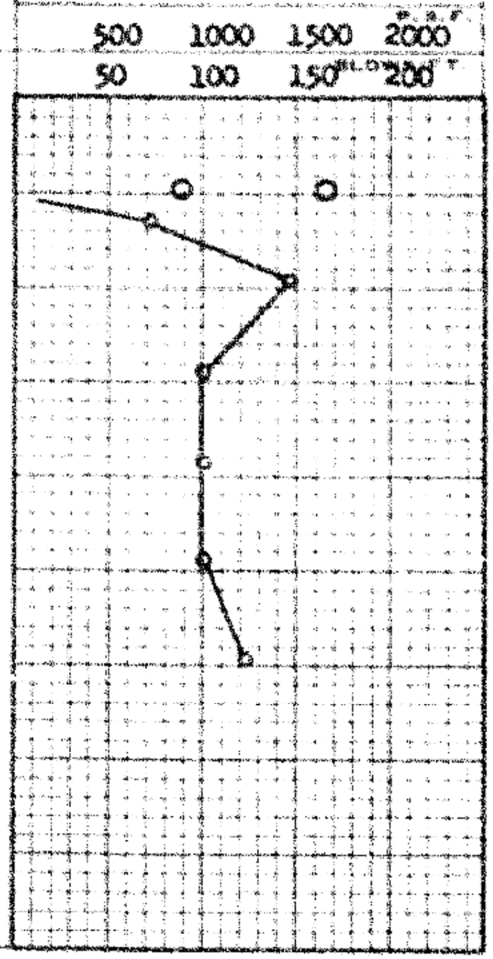
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

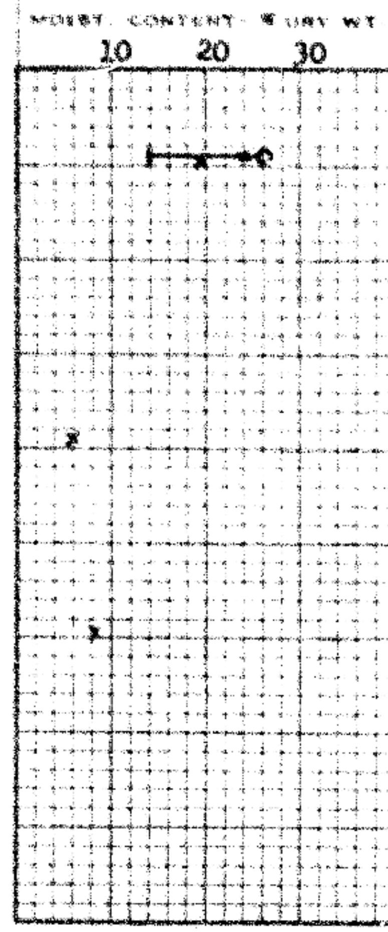
1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Groundlevel	367.0	0
	Granular fill	366.0	
	Dark brown clayey silt (topsoil)	362.0	
	Brown gravelly silty sand	351.0	10
	Grey gravelly silty sand till	337.0	20, 30
	End of borehole		40

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY



SAMPLE	NATURAL UNIT WT. P.C.F.
T1	123.0
S2	131.8
S3	-
S4	-
S5	-
S6	-
S7	-

FORM DB-MC-126 (REV. 1959) - 58-5491

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
JOB 61-F-15
DATUM 369.0'
BORING DATE Mar. 3/61

BORE HOLE NO. 5
STATION See Drawing
COMPILED BY B.K.
CHECKED BY V.K.

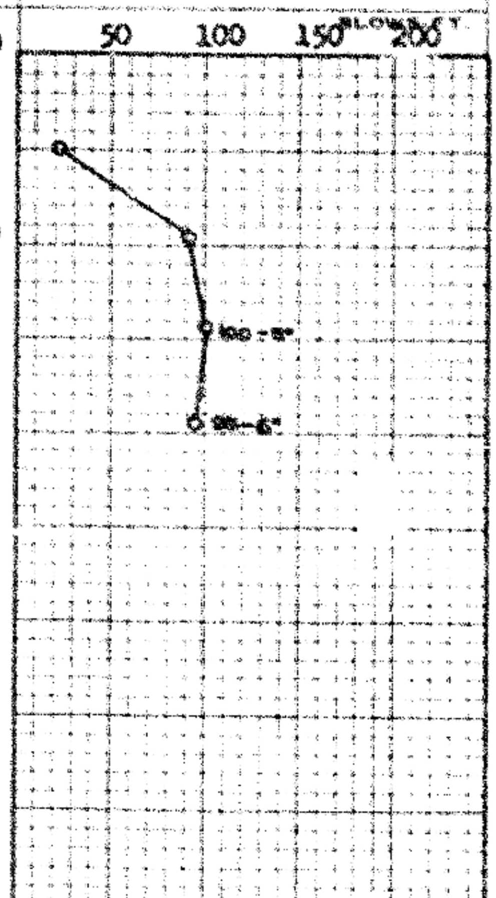
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

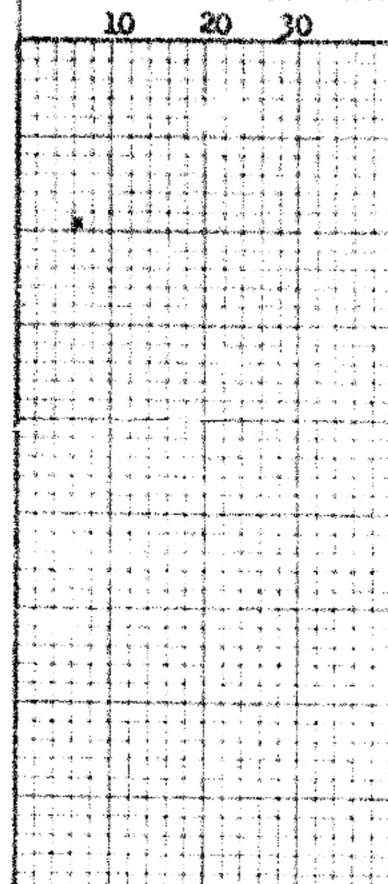
SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	↓ Groundlevel	369.0	0
	Granular Fill	364.5	
	Dark brown clayey silt (topsoil)	362.5	
	Light brown pebbly silty sand (till)		10
		349.5	20
	End of borehole		
			30
			40

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT % DRY WT.



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
S3	-
S4	-

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58
JOB 61-F-15
DATUM 366.0'
BORING DATE Mar. 3/61

BORE HOLE NO. 6
STATION See Drawing
COMPILED BY B.K.
CHECKED BY V.K.

2" DIA SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA CONE
2" SHELBY
CASING

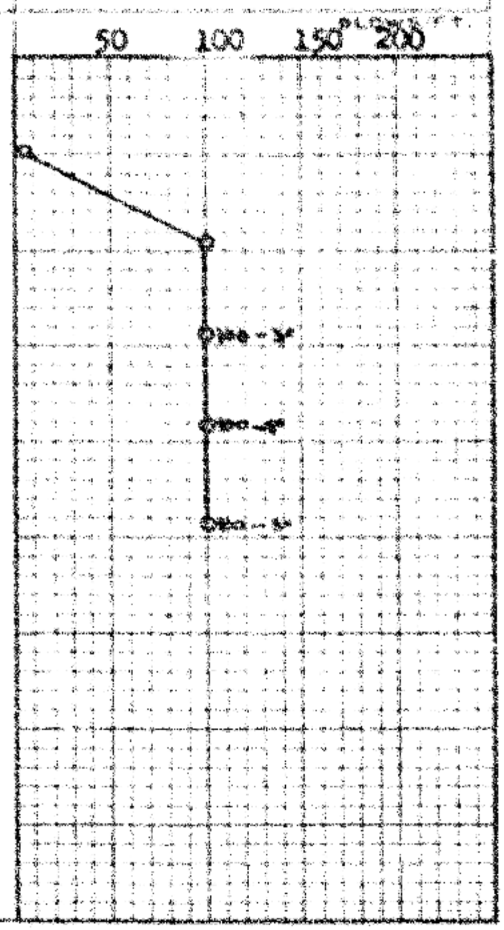
LEGEND

UNCONFINED COMPRESSION (Cu)
VANE TEST (C) AND SENSITIVITY (S)
NATURAL MOISTURE AND LIQUIDITY INDEX
LIQUID LIMIT
PLASTIC LIMIT

DESCRIPTION	ELEV FEET
Groundlevel	366.0
Granular fill	363.5
Dark brown clayey silt (topsoil)	361.0
Light brown pebbly silty sand (till)	353.0
Grey pebbly silty sand (till)	341.7
End of borehole	

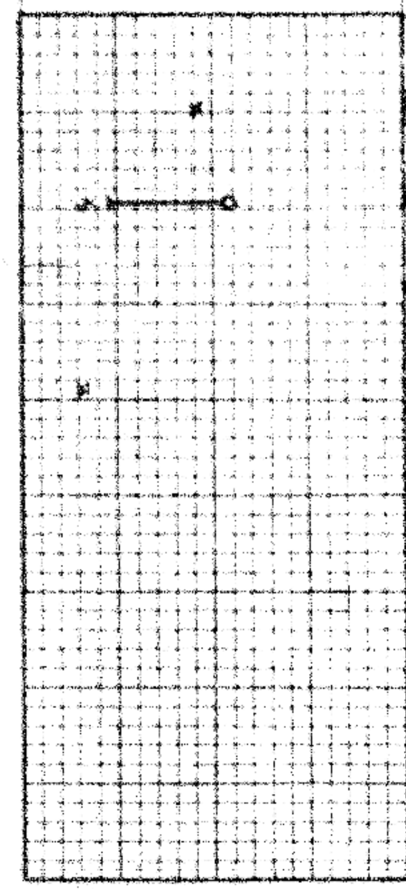
DEPTH FEET: 0, 10, 20, 30, 40

STRENGTH AND PENETRATION RESISTANCE



CONSISTENCY

MOIST. CONTENT % DRY WT



SAMPLE	NATURAL UNIT WT. P.C.F.
S1	-
S2	-
S3	-
S4	-
S5	-

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS AND RESEARCH SECTION

W.P. 118-58 BORE HOLE NO. 7

JOB 61-F-15 STATION See Drawing

DATUM 365.5' COMPILED BY B.K.

BORING DATE Mar. 3/61 CHECKED BY V.K.

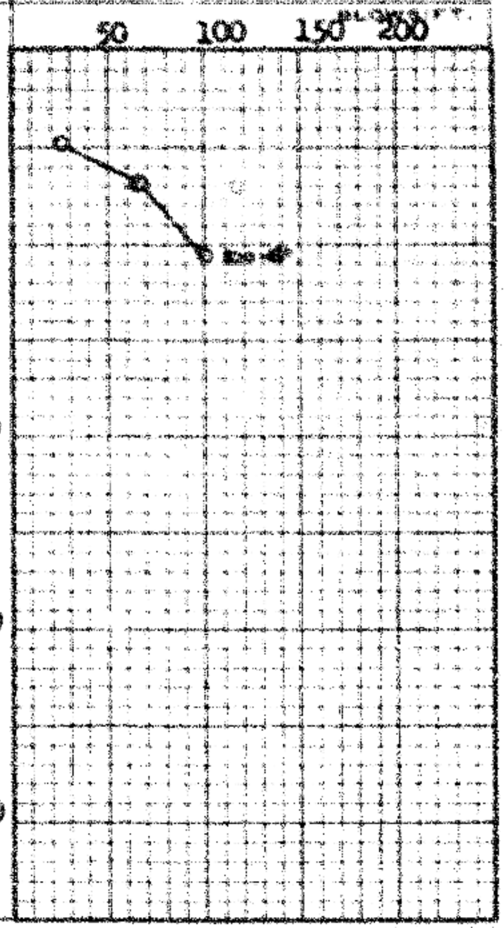
2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
3" SHELBY
CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) O
VANE TEST (C) AND SENSITIVITY (S) +
NATURAL MOISTURE AND LIQUIDITY INDEX LI X
LIQUID LIMIT
PLASTIC LIMIT

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	SAMPLE	NATURAL UNIT WT. P.C.F.
	Groundlevel	365.5	0		
	Granular fill	364.5			
	Dark brown clayey silt (topsoil)	363.5			
	Light brown pebbly silty sand (till)			S1	-
				S2	-
		355.0	10	S3	-
	End of borehole				
			20		
			30		
			40		

STRENGTH AND PENETRATION RESISTANCE P.S.F.



CONSISTENCY

MOIST. CONTENT - % DRY WT.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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