REPORT

March 2012

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CNR Overhead Bridge (Site No. 30-432) Highway 12, District of Midland, Ontario G.W.P. 2004-08-00

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement / rehabilitation of the exiting CNR Overhead Bridge on Highway 12 in the Township of Midland, Ontario.

This report presents the results of the current subsurface investigation carried out for the proposed replacement / rehabilitation of the existing CNR overhead structure, together with the results of the previous foundation investigation carried out in 1971 for the existing structure. The current geotechnical investigation consists of one borehole drilled on Highway 12 near the west abutment of the existing bridge to verify and supplement the results of the previous geotechnical investigation.

This Preliminary Foundation Investigation and Design Report is for planning purposes only and the Design/Build proponent shall satisfy himself as to the sufficiency of the available information and supplement the information as needed to meet the requirement for detail design. The Design/Build proponent is solely responsible for selecting the appropriate foundation alternatives for replacement/rehabilitation of the CNR overhead 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. 2009-E-0100 dated December 2010, and in Section 6.8 of the *Technical Proposal* for this assignment.

2.0 SOURCES OF INFORMATION

The following sources of information were provided to Golder by MH and were reviewed / incorporated into the report where indicated.

- "Foundation Investigation for the Proposed Crossing at the Canadian National Railway and King's Hwy. #12 – Prop. Rev'n Line 'L' ", prepared by Foundations Section, Design Services Branch, Ministry of Transportation and Communications, Job 71-11029, dated July 13, 1971 (for W.P. 650-64-05);
- Drawing No. D-7085-2 titled "C.N.R Overhead – Bore Hole Locations and Soil Strata", prepared by Foundations Section, Design Services Branch, Ministry of Transportation and Communications, dated June 8, 1971;
- Drawing No. D-7085-1 titled "General Layout", Drawing No. D-7085-3 titled "Footing Layout and Details"; Drawing No. D-7085-5 titled "West Abutment"; Drawing No. D-7085-6 titled "East Abutment"; Drawing No. D-7085-7 titled "Retaining Walls"; Drawing No. D-7085-14 titled "Approach Slabs"; and Drawing No. D-7085-17 titled "Standard Details", prepared by Ministry of Transportation and Communications, dated March, 1972.

3.0 SITE DESCRIPTION

3.1 General

The existing CNR Overhead Bridge (Site 30-432) is located approximately 540 m east of Talbot Street along Highway 12 in the County of Simcoe, District of Midland, Ontario. The former CNR tracks ran in a northeast-southwest direction and were originally constructed in a cut approximately 6 m deep, with side slopes ranging from about 1.75 Horizontal: 1Vertical (1.75H:1V) to 2H:1V. As part of the original construction of the overhead structure, the existing grade at the crest of the cut was raised and the approach embankments were constructed on fill soils, up to about 4 m high. Currently, the railway has been decommissioned, the tracks removed, and the corridor is now a recreational trail. In general, the terrain in this area ranges from about Elevation 189 m along the abandoned track/recreational trail to Elevation 197.5 m at the raised Highway 12 road surface. The slopes of the railway cut and the immediately adjacent terrain are grass covered, and strands of trees and shrubs are present throughout the upper/higher ground.

3.2 Site Visit

A site visit was carried out by Golder personnel on August 16, 2011, to assess the present condition of the structure along the roadway, the cut slopes and the ground surface adjacent to the foundations and note any observations of distress potentially attributable to the foundation conditions. From a geotechnical perspective the existing structure and slopes seem to be performing satisfactorily, that is there were no evident signs of settlement or slope instability. However, surficial erosion of the approach embankment slope near the south wing wall at the east abutment and groundwater seepage at the mid-height face of the west abutment front slope were observed. Standing water was present in the ditches paralleling the recreational trail along the toes of the approach embankment cut slopes.

It is noted that reportedly a number of naturally occurring springs were present within the west cut slope during the original investigation at the site in 1971 (MTC, July 1971) and it is understood that mitigation measures (i.e. granular sheeting and sub-drains) were incorporated into the west cut slope design to control water seepage / surficial slope instability issues.

4.0 INVESTIGATION PROCEDURES

4.1 Previous Investigation

The previous investigation carried out by the Ministry of Transportation is summarized in the report titled "Foundation Investigation for the Proposed Crossing at the Canadian National Railway and King's Hwy. #12 – Prop. Rev'n Line 'L' " and consisted of ten boreholes (designated 1 to 10) advanced in the vicinity of the bridge at the approximate locations shown on Drawing 1. Each borehole was accompanied by a Dynamic Cone Penetration Test (DCPT) and one additional DCPT (designated 1A) was driven near the west abutment for a total of eleven DCPT's. The boreholes were advanced using washboring methods and N-size casing by a conventional diamond drill rig adapted for soil sampling. Soil samples were collected using the Standard Penetration Test and the undrained shear strength was measured in cohesive strata using in-situ vane tests. Thin walled (50 mm diameter) "Shelby" tubes were also retrieved in the cohesive deposits. Bedrock was verified by core drilling using a B-size core barrel at one borehole location.

Groundwater conditions across the site were observed in the open boreholes during and immediately following the drilling operations.

The locations of the boreholes were originally surveyed by Station and Offset relative to the Hwy 12 centreline. The approximate borehole locations, referenced to MTM NAD83 co-ordinate system, are provided below and were obtained using the intersection of the Hwy 12 and CN Railway centrelines as a reference point from the previous borehole location plan. The Geodetic elevation of the boreholes at the time of the previous borehole investigation recorded in Imperial Units (feet) and converted to Metric Units (m) is also provided below.

Borehole / DCPT Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole/DCPT Depth (m)
1	4954838.2	279153.2	194.7	9.1
*1A	4954842.3	279139.0	194.8	4.1
2	4954842.7	279130.3	194.7	6.3
3	4954824.9	279149.9	189.9	9.6
4	4954825.6	279168.0	189.9	6.6
5	4954813.3	279163.6	189.2	6.6
6	4954813.1	279182.8	189.2	14.0
7	4954801.0	279196.8	194.0	10.8
8	4954800.8	279178.6	194.8	6.6
9	4954836.9	279176.2	195.1	7.9
10	4954842.8	279108.1	194.8	10.7

*DCPT only

4.2 Current Investigation

The fieldwork for the current subsurface investigation was carried out on July 28, 2011, during which time one borehole (Borehole 11-7) was advanced using a track-mounted CME-55 drill rig, supplied and operated by Davis Drilling Inc. of Milton, Ontario. The borehole location is shown on Drawing 1.

Borehole 11-7 was drilled using 108 mm inner diameter (184 mm outer diameter) hollow stem augers to a depth of 7.1 m below ground surface. Soil samples were obtained at 0.75 m to 1.5 m intervals of depth, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586), driven by an automatic hammer. In situ vane tests (ASTM D2573) were carried out in the cohesive stratum (in an adjacent borehole) to assess the undrained shear strength of the soil.

The groundwater condition was observed in the open borehole during and immediately following the drilling operations. A standpipe piezometer was installed in the borehole drilled adjacent to Borehole 11-7 within which the in situ vane tests were carried out, to permit monitoring of the groundwater level. The piezometer consists of a 50 mm diameter PVC pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled to the ground surface with bentonite pellets in accordance with Regulation 903 (as amended). The details of the piezometer installation and water level readings are indicated on the Record of Borehole sheet contained in Appendix A.

The field work was supervised on a full-time basis by a member of Golder's staff who located the borehole in the field, directed the drilling, sampling, and in situ testing operations, and logged the borehole. 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 applicable ASTM and/or MTO LS procedures.

The surveyed location of the as-drilled borehole (referenced to the MTM NAD83 coordinate system) and ground surface elevation (referenced to Geodetic datum) were provided by MH and are summarized below together with the drilled depth of the borehole.

Borehole Number	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
11-7	4954845.0	279141.3	197.5	7.1

5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

The CNR Overhead site is located within the "Simcoe Uplands" physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[1]. The predominant overburden stratum consists of glacial deposits comprised of sandy silt to silty sand till. Surficial deposits of boulders, sands and silts from the glacial Lake Algonquin overly the glacial till materials. Limestone, dolostone and shale of the Simcoe Group typically underlie the overburden deposits.

The subsurface conditions encountered at the site are generally consistent with the Regional Geology described above.

5.2 Subsurface Conditions

The current subsurface investigation consisted of the advancement of one borehole (Borehole 11-7) on Highway 12 near the west abutment of the existing CNR Overhead structure to confirm and supplement the subsurface information from the previous investigation at the site which consisted of ten boreholes and DCPTs (Boreholes 1 to 10) and one additional DCPT (1A). The locations of the boreholes from the current and previous field investigations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawings 1 and 2. The detailed subsurface soil, rock and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets contained in Appendices A to C. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic sections are inferred from non-continuous sampling and, therefore, represents transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered at the site at the time of the investigation consist of topsoil and fill underlain by a deposit of clayey silt to silty clay. At the Highway 12 approach embankment location, asphalt is present at the road surface and is underlain by granular embankment fill, which in turn is underlain by a deposit of native silty clay. The clayey silt to silty clay is underlain by a deposit of glacial till comprised of silty sand to sandy silt, which in turn is underlain by limestone bedrock. A more detailed description of the major soil deposits encountered in the boreholes is provided in the following sections.

5.2.1 Asphalt

A surficial layer of asphalt (100 mm thick) was encountered in Borehole 11-7.

[1] Chapman, L.J and Putnam, D.F., 1984. The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.

5.2.2 Topsoil

An approximately 150 mm to 300 mm thick layer of topsoil was encountered at the ground surface at the time of the investigation in Boreholes 1 to 3, 5 to 7, and 10. In Boreholes 2, 7, and 10, the topsoil transitioned into a layer of loose brown sand (approximately 300 mm to 600 mm thick).

5.2.3 Fill

The surficial soil in Boreholes 3, 5 and 6 located near the toe of the cut slopes is described as a mixture of topsoil and granular fill, up to 0.9 m thick. A surficial layer of sand to sand and gravel fill was encountered in Borehole 8 and below the asphalt in Borehole 11-7. The sand to sand and gravel fill layer is 0.6 m thick at Borehole 8 and 3.6 m thick in Borehole 11-7.

The SPT "N" values measured in the sand to sand and gravel fill in Borehole 11-7 ranged from 13 to 22 blows per 0.3 m of penetration indicating a compact relative density.

The results of a grain size distribution test completed on one sample of the sand and gravel fill deposit is shown on Figure B1 in Appendix B.

Laboratory natural water contents taken on samples of the sand and gravel fill ranged from 2 per cent to 3 per cent.

5.2.4 Clayey Silt to Silty Clay

A stratum of brown to grey clayey silt to silty clay was encountered below the topsoil in Boreholes 1, 2 and 10, below the sand to sand and gravel fill in Borehole 8 and 11-7, and at the ground surface in Borehole 9. In Borehole 11-7 and 10, the silty clay layer contains interlayers of clay. The top of this deposit ranges from ground surface in Borehole 9 (Elevation 195.1 m) to 3.7 m below ground surface (Elevation 193.8 m) in Borehole 11-7 and the thickness of the deposit ranges from 0.9 m to 2.7 m.

The measured SPT "N" values within the clayey silt to silty clay stratum range from 2 blows to 18 blows per 0.3 m of penetration. In-situ field vane tests carried out within this clayey silt to silty clay stratum measured undrained shear strengths ranging from 30 kPa to greater than 100 kPa. Two triaxial unconsolidated undrained (UU) tests carried out on three samples of this deposit during the previous investigation measured undrained shear strength values ranging from 20 kPa to 75 kPa. The results of the field and laboratory tests indicate that the clayey silt to silty clay stratum is firm to very stiff in consistency.

The natural water content measured on samples of the clayey silt to silty clay stratum range from 32 per cent to 66 per cent.

The results of grain size distribution tests completed on samples of the clayey silt to silty clay are shown on the Record of Borehole sheets in Appendix C.

Atterberg limits testing carried out on six selected samples of this deposit measured plastic limits between 21 per cent and 32 per cent, liquid limits between 38 per cent and 71 per cent, and plasticity indices between 17 per cent and 42 per cent. These test results, which are plotted on Figure B2 in Appendix B and on the plasticity chart included in Appendix C, indicate that the deposit is predominantly a silty clay of intermediate plasticity containing clay interlayers of high plasticity.

5.2.5 Sandy Silt to Silty Sand Till

A glacial till deposit consisting predominantly of brown silty sand to sandy silt, was encountered below the clayey silt to silty clay stratum in Boreholes 1, 2, 8, 9, 10 and 11-7, below the topsoil/fill in Boreholes 3, 5, 6 and 7 and at the ground surface in Borehole 4. The upper 1.7 m portion of the till deposit in Borehole 4 is described as clayey silt. Interlayers/seams of silty sand were encountered in the upper 1.5 m of the glacial till deposit. The presence of cobbles and boulders was recorded on the Record of Boreholes and verified by coring through the obstructions in Boreholes 2 and 5 to 10. The top of the till deposit ranges from ground surface to 6.1 m below ground surface, between Elevation 194.2 m and 188.4 m. Boreholes 1 to 5, 7 to 10 and 11-7 were terminated within this deposit at depths ranging from 6.3 m to 10.8 m below ground surface. In Borehole 6, where the glacial till was fully penetrated, the measured thickness of the till unit is 11.7 m.

The measured SPT "N" values within the silty sand to sandy silt till deposit range from 3 blows per 0.3 m penetration to 100 blows per 0.05 m of penetration. Although very loose/soft to loose/firm layers of silty sand to sandy silt till/clayey silt till were encountered in the upper 1.7 m of the till deposit in Boreholes 3, 4 and 10, the till deposit is generally compact to very dense.

All of the DCPT's advanced in Boreholes 1, 1A, and 2 to 10 during the previous investigation achieved effective refusal (greater than 100 blows per 0.3 m of penetration) within 3 m of penetrating into the glacial till deposit at depths ranging from 1.8 m to 5.5 m below ground surface (Elevation 193.0 m to Elevation 187.3 m).

The natural water content measured on samples of the silty sand to sandy silt till deposit range from 5 per cent to 8 per cent.

The results of grain size distribution tests completed on samples of the silty sand till deposit are shown on Figure B3 in Appendix B and as an envelope developed for the glacial till during the previous investigation in Appendix C.

Atterberg limits testing carried on samples of the glacial till are shown on the plasticity chart in Appendix C and indicate the glacial till deposit is generally a silt till of low plasticity.

5.2.6 Bedrock

In Borehole 6, limestone bedrock was encountered below the glacial till deposit at a depth of 12.5 m below existing ground surface, corresponding Elevation 176.7 m. Bedrock was confirmed by coring 1.5 m into the bedrock using a B-size core barrel and the Total Core Recovery (TCR) was reported as 100 per cent.

5.3 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Boreholes contained in Appendices A and C. A standpipe piezometer was installed in Borehole 11-7 (sealed partially within the silty clay to clayey silt layer and partially within the glacial till deposit) to monitor the groundwater level at the site.

The water levels measured in the open boreholes upon completion of drilling and in the piezometer are summarized below:

Borehole Number	Ground Surface Elevation (m)	Piezometer Installation	Depth to Water Level (m)	Groundwater Elevation (m)	Date
1	194.7	-	(-0.6)*	195.4	April 16, 1971
2	194.7	-	0.7	194.0	April 19, 1971
3	189.9	-	(-6.7)*	196.6	April 21, 1971
4	189.9	-	0.9	189.0	April 26, 1971
5	189.2	-	0.2	189.0	April 27, 1971
6	189.2	-	(-0.6)*	189.8	May 4, 1971
7	194.0	-	1.4	192.6	May 6, 1971
8	194.8	-	1.9	192.9	May 7, 1971
9	195.1	-	5.8	189.3	May 11, 1971
10	194.8	-	1.2	193.6	May 13, 1971
11-7	197.5	Shallow	3.1	194.4	September 30, 2011

*Represents artesian groundwater condition.

Artesian conditions were reported to have been encountered in the lower portion of the silty sand to sandy silt till deposit during the drilling of Boreholes 1, 3 and 6 between Elevations 176.8 m and 186.2 m. At the time of drilling, artesian heads of 0.6 m above ground surface were measured in Boreholes 1 and 6 while a head of 6.7 m above ground surface (Elevation 196.6 m) was measured in Borehole 3.

During the drilling of Borehole 9, water loss was reported to have occurred at Elevation 187.1 m (within the glacial till) suggesting that a more pervious zone within the till is present at this elevation.

The groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

6.0 CLOSURE

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

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CNR OVERHEAD BRIDGE, HIGHWAY 12
DISTRICT OF MIDLAND, ONTARIO
G.W.P. 2004-08-00

7.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

7.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement/rehabilitation of the existing CNR overhead structure. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current and previous subsurface investigation. The discussion and preliminary 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 preliminary design of the project, and for which, ultimately, provision will have to be made at the detail investigation design stage of the project and as the contract documents are prepared.

This Preliminary Foundation Investigation and Design Report is for planning purposes only and the Design/Build proponent shall satisfy themselves as to the sufficiency of the information and supplement the information as needed to meet the requirements for detail design. The Design/Build proponent is solely responsible for selecting the appropriate foundation alternatives for replacement/rehabilitation of the CNR Overhead structure and any associated retaining walls.

7.2 Foundation Options

The former CNR track bed and present recreational trail are oriented in a northeast-southwest direction, located in a depressed corridor originally constructed as a deep cut (up to about 6 m deep) with side slopes ranging from about 1.75H:1V to 2H:1V at the CNR Overhead structure location. The current Hwy 12 road grade and approach embankments are constructed on a partial fill (up to about 4 m high) at the crest of the cut slope based on the original design drawings provided by MH (Drawing No. D-7085-1 “General Layout” dated March 1972). The existing three-span structure carries Highway 12 over the abandoned CNR recreation trail (former track bed) and is approximately 70 m long and 11 m wide. The abutments and piers are supported on spread footings founded within the compact to very dense silty sand to sandy silt till deposit at about Elevation 192.0 m and 186.6 m, respectively. According to the design drawings, a Granular ‘A’ fill blanket and toe drains were placed on the front slope at the west abutment location to control water seepage within the cut slopes.

We understand that the existing structure is to be replaced or rehabilitated to accommodate future Highway 12 widening and to allow passage of the recreational trail below the highway. According to the Preliminary General Arrangement drawing (dated August 16, 2011) provided by Morrison Hershfield, the existing overhead bridge is to be removed and replaced with a cast-in-place concrete box culvert embedded within new embankment fill. The proposed culvert is approximately 30 m long, 8 m wide and 5.2 m high, with a proposed soil cover up to about 4 m thick to the level of the highway subgrade. A precast box culvert is not considered practical due to the size and weight of the 8 m wide culvert segments. In addition, due to MTO right-of-way constraints, we understand that the new highway embankment side-slopes from the roadway level to the ends of the culvert are proposed to be as steep as 1.6H:1V and supported by retaining walls at/near the toes of the proposed embankment fills in some areas.

Based on the subsurface conditions, the following preliminary foundation options have been considered for the replacement / rehabilitation of the existing bridge structure. A summary of the advantages, disadvantages, relative costs and risks associated with each option is provided in Table 1 following the text of this report.

- **Box culvert founded on the dense to very dense silty sand to sandy silt till (full replacement of bridge structure):** A box culvert founded on the dense to very dense silty sand to sandy silt till at or slightly below the existing ground surface is feasible for the replacement of the existing bridge structure. If encountered, any unsuitable surficial topsoil/ballast/backfill materials can be removed or subexcavated and replaced with competent engineered fill in order to provide adequate geotechnical resistances.
- **Open footing culvert or footings for new bridge structure founded on the dense to very dense silty sand to sandy silt till (full replacement of bridge structure):** Strip footings for an open culvert or spread footings for a new bridge structure founded on the dense to very dense silty sand to sandy silt till (1.5 m to 2 m below the existing ground surface) is feasible for the replacement of the existing bridge. The potential for temporary slope instability during excavation for spread / strip footings exists due to the current slope configuration and artesian groundwater conditions present within the glacial till. If this option is considered, the artesian groundwater conditions should be re-assessed and proper staging or dewatering during foundation construction must be undertaken during detail design.
- **Steel H-piles driven to found in the silty sand to sandy silt till (new bridge structure):** Steel H-piles driven to refusal into the silty sand to sandy silt till are feasible foundation elements to support the replacement bridge, however, driving piles through the till containing cobbles and boulders will be difficult and pre-augering into the till to achieve the minimum pile length for integral abutment conditions is likely required. Additionally, there is a potential for upward flow of groundwater and migration of fine soil particles due to the artesian groundwater conditions present at the site. This option is not considered practical.
- **Caissons founded in the silty sand to sandy silt till (new bridge structure):** Caissons founded in the silty sand to sandy silt till are feasible foundation elements to support the replacement bridge. Due to the presence of water-bearing cohesionless interlayers/seams encountered throughout the till deposit and presence of artesian groundwater conditions, temporary or permanent liners would be required to maintain the caisson hole open, but there is still a risk of loosening the soils at the base of the caisson. In addition, caisson installation through the till containing cobbles and boulders will be difficult. This option is not considered practical
- **Box culvert/open footing culvert supported on deep foundations (full replacement of bridge structure):** Although feasible, a box or open footing culvert supported on deep foundations are not considered practical, nor necessary, due to the presence of competent founding soils at or below the existing ground surface.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to replace the existing structure with a concrete box culvert founded on the dense to very dense silty sand to sandy silt till at or below the existing ground surface. Preliminary design recommendations for the concrete box and open footing culvert options are provided below.

7.3 Box Culvert Replacement

7.3.1 Founding Elevation and Geotechnical Resistance

The preferred preliminary foundation alternative is a concrete box culvert founded slightly below the existing ground surface within the dense to very dense silty sand to sandy silt till at or below Elevation 188 m. Any existing topsoil, fill, clayey silt or silt layers, or loosened / disturbed soils at the proposed founding level should be removed to a subgrade of dense to very dense till. The actual founding level will depend on the final location of the culvert within the width of the former railway track bed, thickness of the base slab of the culvert, ground surface invert level within the culvert, frost protection considerations, the depth of granular bedding / levelling pad beneath the culvert, and the results of additional boreholes at the actual culvert location to be drilled during detailed design. Consideration must also be given to temporary slope instability issues during construction if the founding level is lower than Elevation 188 m due to the artesian groundwater conditions and current slope configuration of the depressed corridor.

Assuming a minimum 8 m wide box culvert, founded in the dense to very dense glacial till at or slightly below the founding level described above (Elevation 188m), a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 500 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 300 kPa (for 25 mm of settlement) may be used for preliminary design.

The preliminary geotechnical resistances should be reviewed and revised as necessary if the selected footing width or founding elevations differ from those given above and based on the results of the detailed geotechnical investigation. The preliminary geotechnical resistances provided above are for loads applied perpendicular to the surface of the footings and, 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.

Consideration could be given to using a lower geotechnical resistance for preliminary design given that the proposed culvert foundation footprint is located in close proximity to the existing piers and, based on the original design drawings, a portion of the box culvert foundations may end up being supported on the existing pier foundation backfill material, the quality of which is not known. In addition, it is likely that ballast material (i.e. gravel to cobble-sized rock fill) is present below the ground surface of the recreational trail at and below the proposed culvert founding level which may or may not be suitable as a foundation material. The suitability of the ballast and/or existing pier backfill to support the culvert will need to be assessed during the detail design investigation and the geotechnical resistance values confirmed or revised as necessary.

As an alternative, any fill materials or unsuitable soils that may be encountered during detail design could be subexcavated to the level of a dense to very dense till subgrade and replaced with engineered fill to the proposed/design founding level of the base of the culvert. The width of the required subexcavation should be defined by lines extending from 0.3 m beyond the outside edges of the proposed culvert base, outward and downward at 1H:1V. A minimum 8 m wide box culvert founded on properly placed and compacted engineered fill (i.e. Granular ‘A’ or ‘B’ Type II) supported on the dense to very dense glacial till can be designed based on a factored geotechnical axial resistance at ULS equal to 500 kPa and geotechnical resistance at SLS equal to 300 kPa (for 25 mm of settlement) for preliminary design.

The bottom of the base slab should be founded at a minimum depth of 1.6 m below the lowest surrounding grade to provide frost protection, as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

The box culvert subgrade should be inspected by a Quality Verification Engineer (QVE) following subexcavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to ensure that all existing fill and any surficial organic soils or other unsuitable material have been removed.

The founding soils will be susceptible to disturbance and potential loosening/softening on exposure to water and construction traffic. Granular bedding, consisting of SP 110S13 (Aggregates) Granular 'A' or 'B' Type II (minimum 200 mm thick), should be placed on the native soil or engineered fill (below the culvert base slab) immediately following subgrade preparation. Alternatively, a concrete working slab (minimum 100 mm thick with a minimum compressive strength of 20 MPa) should be placed on the approved subgrade to protect it from degradation. In this case, a 75 mm thick layer of SP110513 Granular 'A' or concrete fine aggregate meeting the gradation requirements set out in OPSS 1002 (Aggregates – Concrete) should be placed on top of the concrete working slab to provide a "levelling pad" for the box culvert replacement.

7.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete box culvert and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. For cast-in-place concrete base constructed on a granular bedding material or granular levelling pad, the coefficient of friction, tan Φ' can be taken as 0.5. If pre-cast box culvert sections are to be used, the coefficient of friction, tan δ, can be taken as 0.45.

7.4 Open Footing Culvert Replacement / New Retaining Walls

7.4.1 Founding Elevation and Geotechnical Resistance

Strip footings for an open footing culvert replacement and for any associated concrete wing walls/retaining walls, should be founded at a minimum depth of 1.6 m below the lowest surrounding grade to provide adequate protection against frost penetration as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario). The footings should extend below any existing fill and surficial organic materials, where present.

For the proposed culvert replacement, strip footings founded at least 1.6 m below the adjacent ground surface within the dense to very dense silty sand to sandy silt till at or slightly below Elevation 188 m may be assumed for preliminary design. Consideration must also be given to temporary slope instability issues during subexcavation to founding levels lower than Elevation 188 m due to the artesian groundwater conditions and current slope configuration of the depressed corridor. Although the existing footings at the piers are founded at Elevation 186.6 and show no signs of settlement or instability, the potential for instability for excavations to this level or lower should be investigated further during detail design. Given the close proximity of the existing pier foundations to the proposed open footing culvert foundations, this option may not be feasible as it is assumed the existing pier foundations will remain. If the existing pier foundations are to be removed, the potential impact (i.e potential disturbance) to the founding soils at the proposed open culvert footings needs to be investigated during detail design. Assuming a minimum 2 m wide strip footing founded within the dense to very dense silty sand to sandy silt till at or below Elevation 188m, a factored geotechnical axial resistance at Ultimate Limit State

(ULS) of 400 kPa and a geotechnical resistance at Serviceability Limit State (SLS) of 250 kPa (for 25 mm of settlement) may be used for preliminary design.

For the proposed retaining walls located at the toe of the new embankment fill front slopes near the ends of the proposed culvert, strip footings founded at least 1.6 m below the adjacent ground surface and within the compact to very dense silty sand to sandy silt till may be assumed for preliminary design. Consideration must also be given to temporary slope instability issues and groundwater seepage issues during subexcavation for footing construction, especially if the retaining wall footings are to be founded within the glacial till along the west cut slope where previous groundwater springs were identified and granular sheeting/sub-drains have been incorporated into the construction of the current slope to mitigate slope instability due to the artesian groundwater conditions. Assuming a minimum 3 m wide strip footing founded below frost depth within the compact to very dense glacial till as described above, a factored geotechnical axial resistance at Ultimate Limit States (ULS) of 400 kPa and a geotechnical resistance at Serviceability Limit State (SLS) of 250 kPa (for 25 mm of settlement) may be used for preliminary design.

The preliminary geotechnical resistances provided above should be reviewed and revised as necessary if the selected footing width or founding elevations differ from those given above (and when founding elevations for the retaining walls are known) and based on the results of the detailed geotechnical investigation. These preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings and where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the CHBDC (2006) and its Commentary.

7.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. For cast-in-place concrete footings constructed on the compact to very dense silty sand to sandy silt till deposit, the coefficient of friction, tan Φ' can be taken as 0.5.

7.5 Deep Foundations

Given that the near surface subsoils are competent, the presence of cobbles and boulders in the sandy silt to silty sand till, and artesian groundwater pressures within the till, deep foundations (piles or caissons) are not considered to be practical options for this site.

7.6 Seismic Considerations

7.6.1 Site Coefficient

The soil profile at this site has been classified as Type I according to the CHBDC. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design, if required.

7.6.2 Seismic Analysis Coefficient

The potential for seismic (earthquake) loading must also be considered for the design of retaining walls in accordance with Section 4.6 of the CHBDC. According to Table A3.1.1 of the CHBDC, this site is located in Seismic Performance Zone 1. In accordance with Section 4.4.5.1 of the CHBDC, seismic analysis is not required for structures located in Seismic Performance Zone 1.

7.7 Retaining Walls

According the preliminary GA drawing, retaining walls up to about 5 m high are proposed near the ends of the culvert to maintain the footprint of the new embankment fill within the existing MTO right-of-way. Feasible retaining wall options include:

- Concrete retaining walls supported on spread footings designed on the basis of the geotechnical resistance values provided in Section 7.4 may be used for preliminary design.
- Retained Soil System (RSS) walls are feasible and could be incorporated into the overall design of a composite reinforced slope/wall system. Subexcavation of any surficial soft/loose materials, where encountered, and replacing with compacted granular material, will be required to allow for construction of the wall foundation and reinforced soil mass. Typically, an RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall mass. For preliminary design, the geotechnical resistance values provided in Section 7.4 for shallow foundations may be used. As the reinforced earth structure is a proprietary system, it is noted that it is the responsibility of the wall designer to ensure that the internal stability of the wall is adequate. Given that the underlying foundation soils consist of compact to very dense silty sand to sandy silt till, the global stability of the embankment/wall system is not anticipated to be a concern, however, this will need to be confirmed during detail design.

7.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the box culvert walls and any associated 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 surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. As discussed in Section 7.6, seismic (earthquake) loadings are not anticipated to be required for this structure.

The following recommendations are made concerning the design of the box culvert walls and associated retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of SP 110S13 (Aggregates) Granular "A" or Granular "B" Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the box culvert walls and retaining 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 sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill) and 3121.150 (Walls, Retaining, Backfill).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.6 m behind the back of the walls (in accordance with Figure C6.20(a) of the Commentary to the CHBDC).

For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC*)*.

- For restrained or unrestrained structures, the pressures are based on the proposed embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used assuming the use of granular or earth fill:

	Granular "A"	Granular "B" Type II	Earth Fill
Soil unit weight:	22 kN/m^3	21 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	 0.33 0.50

- For unrestrained structures such as the proposed retaining walls at the toe of the proposed embankment fill slopes where the embankment fill above the top of the wall is sloped at 1.6H:1V, the active lateral earth parameters (unfactored) will increase and the following values may be used for preliminary design.

	Granular "A"	Granular "B" Type II	Earth Fill
Coefficients of static lateral earth pressure: Active, K_a	 0.48	 0.48	 0.58

The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

7.9 Subgrade Preparation and Embankment Construction

Embankment fill up to 8 m thick is anticipated adjacent to the culvert structure to fill the gap between the existing ground surface near the recreational trail and the existing Highway 12 road surface. The thickness of the fill will reduce east and west of the recreational trail as the ground surface of the existing cut slopes (i.e. existing depressed corridor) rises to meet the Highway 12 road surface at the existing east and west abutment.

It is recommended that all topsoil, softened/loosened soils or fill materials, and soils containing organics, be stripped from the proposed embankment footprint. Consideration should be given to leaving the existing granular layer in place on the west cut slope to reduce the potential for increasing groundwater seepage along the cut slope face during embankment fill placement. The subgrade should be proof-rolled, where possible, prior to fill placement to identify any loose/softened areas requiring subexcavation or additional compaction. Embankment fill should consist of suitable earth fill placed and compacted in accordance with OPSS 501 (Compacting) and 206S03 (Earth Excavation and Grading). Backfill to the culvert against the existing cut slopes should be placed consistent with OPSD 208.010 (Benching of Earth Slopes).

The preliminary assessment of settlement of the foundation soil (i.e. the dense to very dense silty sand to sandy silt till) below the culvert and embankment fill is expected to be negligible and will occur relatively quickly, during and immediately following construction, based on the nature of the soils at the site. Settlement of the embankment fill itself is expected to be less than 25 mm provided that the granular fill is properly placed and compacted. Additional settlements should be expected if cohesive fill or rock fill is used.

If rock fill is being considered, it is generally anticipated that total settlements would be in the order of about 0.75 per cent (60 mm) for a total rock fill thickness of approximately 8 m. If rock fill is used as cover for the culvert, the estimated total settlement is approximately 20 mm for a total rock fill thickness of about 4 m. Approximately 90 percent of the total settlement of the rock fill is anticipated to occur within the first year after construction and would result in a differential settlement of approximately 40 mm, requiring timely maintenance of the roadway surface. Therefore, the use of rock fill is not recommended.

Side-slopes no steeper than 2H:1V are recommended for conventional engineered fill embankment construction using suitable earth/granular fill. In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided (for conventional 2H:1V slopes) where the embankment side slope height is equal to or greater than 8 m, such that the uninterrupted slope height does not exceed 8 m. To reduce the potential for erosion of the embankment side slopes (2H:1V or shallower) due to surface water run-off and to establish vegetation on the slopes, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the new embankment. Topsoil should be placed on exposed granular fill slopes in accordance with OPSS 802 (Topsoil) and covered with erosion protection in accordance with OPSS 804 (Seed and Cover) or pegged sod in accordance with OPSS 803 (Sodding).

Due to construction having to be constrained to within the right-of-way at the site, side-slopes as steep as 1.6H:1V in combination with retaining walls are shown on the preliminary GA drawing. Steeper side-slopes can be achieved using a Reinforced Soil Structure (RSS) in combination with retaining walls. It is noted that the design of the RSS slope will need to be carried out by the proprietary product supplier to ensure that the internal stability of the slope is adequate, and to ensure proper erosion protection and vegetation cover is provided.

The preliminary assessment for the global stability of a conventional embankment fill or steepened RSS slope/wall system was calculated based on limit equilibrium analyses using the commercially available program SLIDE, 2005 (Version 5.018) produced by Rocscience Inc. employing the Morgenstern-Price method of analyses. A factor of safety greater than 1.3 under static conditions was calculated, assuming proper subgrade preparation and placement and compaction of embankment fill materials. The preliminary assessment of the stability of the embankments should be reviewed and confirmed based on the results of the detail investigation. Mitigation measures to improve slope stability, if required, can be implemented, such as by utilising light weight fill materials.

7.10 Construction Considerations

The following subsections identify potential construction issues that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during the design build stage for incorporation in the Contract Documents.

7.10.1 Excavation and Temporary Protection Systems

Temporary subexcavation for the construction of the box culvert and retaining wall foundations are expected to extend through the existing trail pavement surface, fills (including possible ballast material), clayey silt, and into the compact to very dense silty sand to sandy silt till deposit. The excavations will also extend below the groundwater level, which was measured to be typically at about ground surface (Elevation 189.2 m) in the boreholes closest to the recreational trail but measured to be artesian and as high as Elevation 196.6 m in boreholes advanced in the area.

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, clayey silt soils, and sandy silt to silty sand till soils would be classified as Type 3 soil according to the 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.

Given that the existing cut slopes of the recreational trail corridor have exhibited and continue to exhibit evidence of groundwater seepage, preliminary slope stability analyses were carried out assuming the artesian groundwater conditions are present at the site. The slope stability analyses indicate that a Factor of Safety equal to 1.3 against global instability is achieved for the existing configuration at the west abutment front slope. The proposed excavation for the proposed culvert is shown on the General Arrangement drawing to extend about 1.6 m below ground surface, which results in a temporary Factor of Safety against global instability equal to about 1 suggesting potential for slope instability during temporary excavation operations. The existing spread footings at the piers are shown to be founded about 2.3 m below existing ground surface and although no information is available to suggest that special construction techniques were required (e.g. dewatering or depressurization of the underlying artesian conditions) or difficulties encountered during the construction of the existing pier foundations at the site, excavations for the new culvert foundations should be kept as shallow as possible and/or staged excavation should be considered to avoid excessive groundwater seepage discharging into the excavation and slope instability occurring.

According to the preliminary GA drawings, temporary roadway protection may be required near the centreline of Highway 12 as part of the staging process to allow for live traffic to be transferred to a portion of the new embankment fill while the remaining half of the bridge deck is removed and the remaining half of the embankment fill is placed up to the existing Highway 12 subgrade level. The temporary support system 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.

7.10.2 Groundwater Control

The excavations for culvert and retaining wall foundations will extend below the groundwater table typically measured to be at about ground surface (Elevation 189.2 m) in the boreholes closest to the recreational trail. In addition, artesian groundwater conditions up to 6.7 m above ground surface (Elevation 196.6 m) were present in Borehole 6, located near the northeast limit of the proposed culvert. Groundwater seepage was observed at the west abutment front slope and standing water was present in the ditches along the toe of slope of the current recreational trail during a site visit in 2011.

Although groundwater levels are measured to be relatively high relative to the proposed culvert / wall foundation level, it is expected that groundwater seepage from the compact to very dense silty sand to sandy silt till or perched on top of the existing topsoil, fill, native clayey soils is expected to be effectively handled by modifying existing or constructing new ditches and/or pumping from filtered sumps placed at the base of the excavations. However, it is recommended that the founding level of the culvert base, or culvert strip footings, be designed as high as possible and the extent and depth of any topsoil, existing fills (possibly ballast), or unsuitable founding soils be determined during detailed design. After confirmation of subexcavation depths for detail design, the extent of groundwater control efforts should be re-assessed and modified as necessary.

In addition, a sand drain, with or without a perforated drain pipe, should be considered in the design of the culvert backfill along the toe of the existing slopes and/or at the existing ditches to dissipate groundwater seepage as noted emanating from the face of the slope(s) and to maintain proper drainage of the site.

7.10.3 Subgrade Protection

The silty sand to sandy silt till soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic, ponded water, and/or groundwater seepage. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing or an NSSP included in the Contract Documents during detail design.

7.10.4 Obstructions

The soils at this site are glacially derived and as such will contain cobbles and boulders, as noted in the borehole records. The presence of the cobbles and boulders may affect subexcavation for shallow footings and/or the installation of deep foundations. It is recommended that a Non Standard Special Provision (NSSP) be included in the Contract Documents during detail design to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstructions.

8.0 CLOSURE

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

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GA/KJB/JMAC/jl/sm

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REFERENCES

Canadian Geotechnical Society, 2006. Canadian Foundations Engineering Manual. 4th Edition.

Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code* and *Commentary on CAN/CSA S6 06*. CSA Special Publication, S6.1 06.

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.

Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification For Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 1002	Material Specification for Aggregates - Concrete

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
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

Construction Design Estimating and Documentation (CDED) Special Provisions (SP)

SP 110S13	Material Specification for Aggregates - Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Earth Excavation, Grading

ASTM International

ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Test in Cohesive Soil
ASTM D1587	Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes

Table 1 –Comparison of Foundation Alternatives
Highway 12 / CNR Overhead Structure Replacement
G.W.P. 2004-08-00

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Box culvert founded on the dense to very dense silty sand to sandy silt till at or slightly below the existing ground surface (full replacement of bridge structure)	• Requires very little excavation and hence reduced risk of loosening of subgrade soils due to artesian ground water pressures • Conventional construction • Adequate foundation resistances available at/near existing ground surface • Relatively minor groundwater seepage anticipated from sandy silt to silty sand till deposit can be handled by pumping from sumps	• Lower geotechnical resistances as compared with deep foundations, but adequate for a culvert design • Requires fill material (earth or rock fill) to fill-in present gap between abutments of existing structure	• Least expensive option	• Lower geotechnical resistances or possible replacement with engineered fill if poor surficial soils / ballast / backfill materials are encountered at the proposed elevation of the culvert • End slopes may extend beyond MTO right of-way
Open footing culvert or footings for new bridge structure founded on the dense to very dense silty sand to sandy silt till 1.5 m to 2 m below the existing ground surface (full replacement of bridge structure)	• Existing piers and abutments supported on strip foundations have performed well therefore new strip footing foundation could be founded at same level • Conventional construction	• Lower geotechnical resistances as compared with deep foundations • Potential for encountering groundwater artesian conditions during excavation operations	• Less expensive than deep foundations but potentially more expensive due to footing construction, especially for a new bridge structure	• Potential for slope instability due to excavations to depths of about 1.6 m to 2 m below the existing ground surface. • May interfere with existing pier spread footings

Foundation Option	Advantages	Disadvantages	Relative Costs	Risks
Steel H-piles driven to found in the silty sand to sandy silt till (new bridge structure)	• Allows for integral abutment construction • Higher resistances available than for shallow foundations	• Difficulties driving through the very dense till with cobbles/boulders • Pre-augering likely required to achieve minimum pile lengths • Requires pile cap below ground surface; potential for encountering groundwater artesian conditions during excavation operations	• High costs due to difficulties driving through cobbles/boulders and/or requirement for preaugering. • Dewatering costs	• Difficult driving through very dense till containing cobbles and boulders • Due to artesian conditions at the site, potential risk of upward flow of water and fine soil particles along pile shaft; mitigation measures may be required
Caissons founded in the silty sand to sandy silt till (new bridge structure)	• Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles • No excavation required for pile cap	• Difficult augering through cobbles/boulders; likely requires steel liners • Does not allow for integral abutment design • Steel liner will be required to allow for advancing caissons below groundwater level	• High costs but higher capacity will result in fewer caissons	• Water-bearing cohesionless deposits could contribute to loss of ground; temporary or permanent liners would be required; likely not possible to inspect caisson base and risk of loosening at base. • Drilling must be advanced through till containing cobbles and boulders
Box culvert /open footing culvert on deep foundation	Not required/Not practical			

MINISTRY OF TRANSPORTATION, ONTARIO

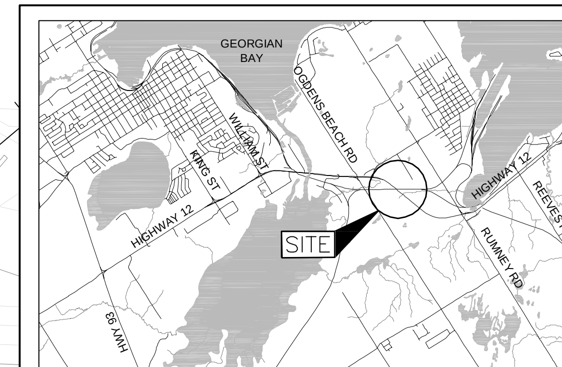
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2004-08-00

SHEET

HIGHWAY 12
CNR OVERHEAD STRUCTURE
BOREHOLE LOCATIONS

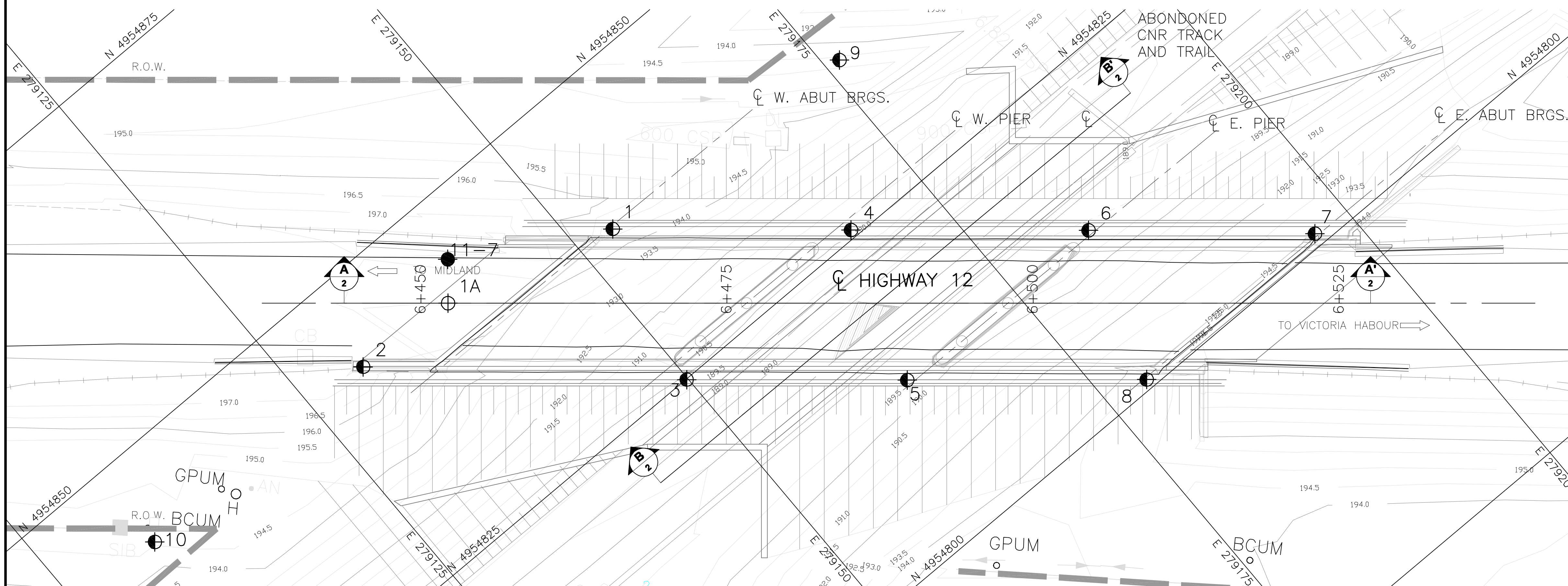
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole – Current Investigation
- Borehole and DCPT – Previous Investigation
- Dynamic Cone Penetration Test (DCPT) – Previous Investigation



PLAN
SCALE
4 0 4 8 m

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	194.7	4954838.2	279153.2
1A	194.8	4954842.3	279139.0
2	194.7	4954842.7	279130.3
3	189.9	4954824.9	279149.9
4	189.9	4954825.6	279168.0
5	189.2	4954813.3	279163.6
6	189.2	4954813.1	279182.8
7	194.0	4954801.0	279196.8
8	194.8	4954800.8	279178.6
9	195.1	4954836.9	279176.2
10	194.8	4954842.8	279108.1
11-7	197.5	4954845.0	279141.3

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

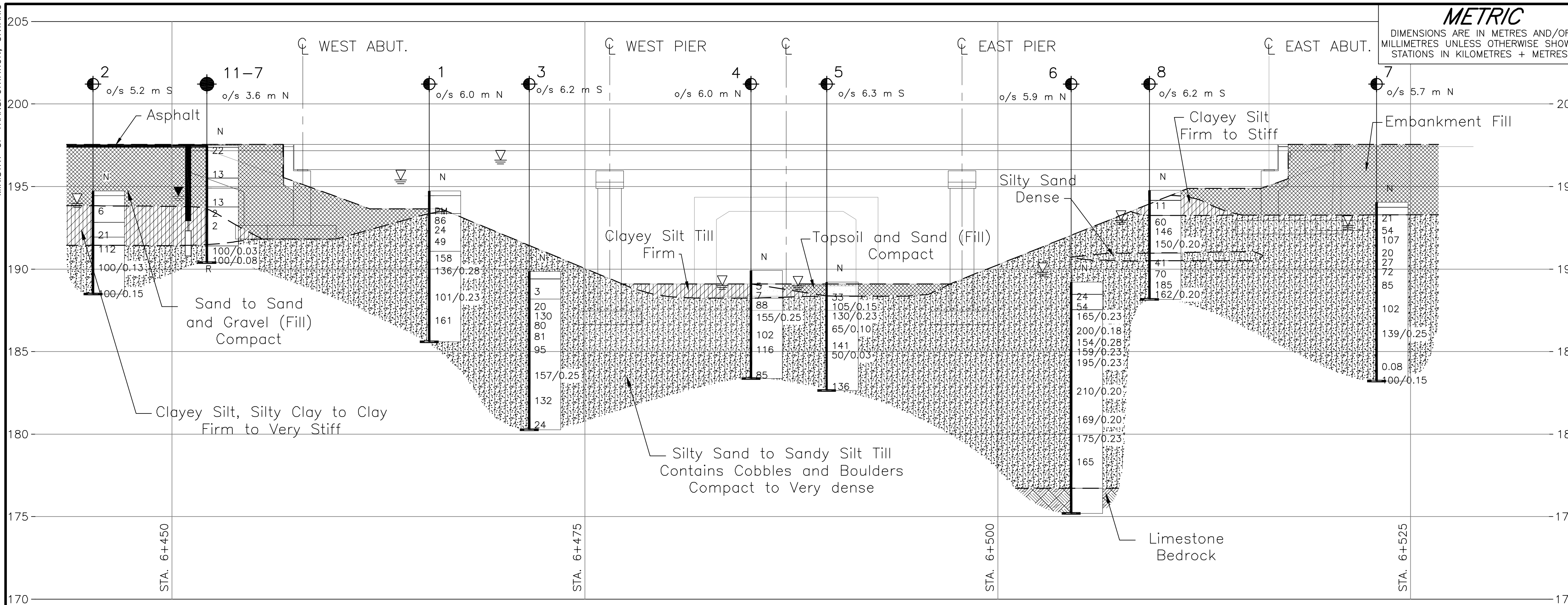
Base plans provided in digital format by Morrison Hershfield, drawing file nos. 30432-01(5.6m Opening).dwg, CNR Overhead-Contour.dwg, x104178Align-CNR_Ovp.dwg and x104178Base_CNR_Ovp.dwg, received October 24, 2011.

NO.	DATE	BY	REVISION

Geocres No. 31D-536

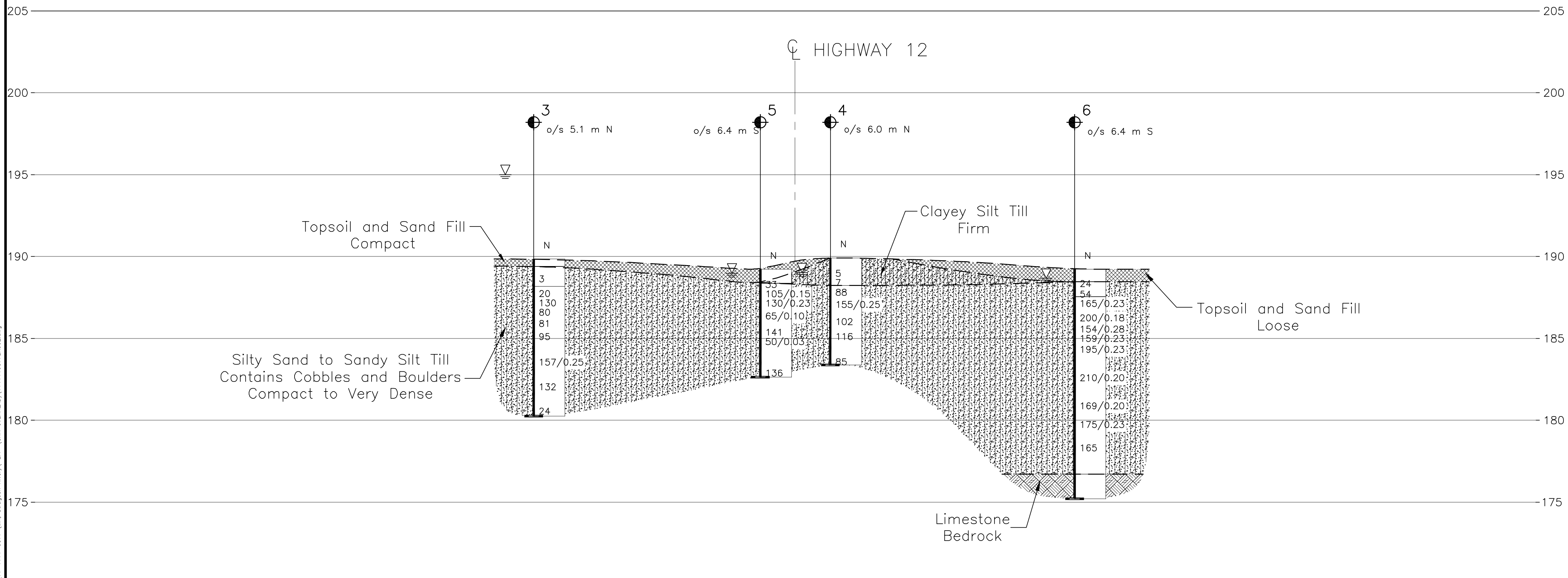
HWY. 12	PROJECT NO. 11-1111-0077		DIST.
SUBM'D. GA	CHKD. KJB	DATE: 3/27/2012	SITE: 30-432
DRAWN: CD	CHKD. GA	APPD.	DWG. 1

PLOT DATE: March 27, 2012
FILENAME: T:\Projects\2011\11-1111-0077 (MH, Sturgeon River)\-BA- (CNR OVERHEAD)\1111100778A001.dwg



A-A' 1 HIGHWAY 12 CENTRELINE PROFILE

SCALE 3 0 3 6 m



B-B' 1 ABANDONED CNR TRACK AND TRAIL CENTRELINE PROFILE

SCALE 3 0 3 6 m

METRIC

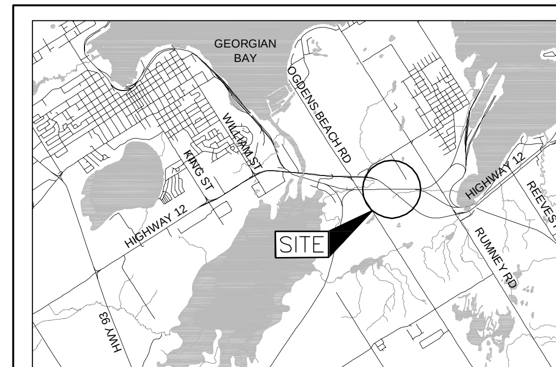
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2004–08–00

HIGHWAY 12
CNR OVERHEAD STRUCTURE
SOIL STRATA

SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE 1.5 0 1.5 3 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on Sept. 30, 2011
- WL upon completion of drilling – Previous Investigation 1971
- R Refusal

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
1	194.7	4954838.2	279153.2
2	194.7	4954842.7	279130.3
3	189.9	4954824.9	279149.9
4	189.9	4954825.6	279168.0
5	189.2	4954813.3	279163.6
6	189.2	4954813.1	279182.8
7	194.0	4954801.0	279196.8
8	194.8	4954800.8	279178.6
11–7	197.5	4954845.0	279141.3

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 plans provided in digital format by Morrison Hershfield, drawing file nos. 30432–01(5.6m Opening).dwg, CNR Overhead–Contour.dwg, x104178Align–CNR_Ovp.dwg and x104178Base_CNR_Ovp.dwg, received October 24, 2011.

NO.	DATE	BY	REVISION

Geocres No. 31D–536

HWY. 12		PROJECT NO. 11–1111–0077	DIST.
SUBM'D. GA	CHKD. KJB	DATE: 3/27/2012	SITE: 30–432
DRAWN: CD/JFC	CHKD. GA	APPD.	DWG. 2

MINISTRY OF TRANSPORTATION, ONTARIO

APPENDIX A

Record of Boreholes, Current Investigation

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
in 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
υ	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
μ	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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_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	over-consolidation ratio = σ'_p / σ'_{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

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

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 11-7

SHEET 1 OF 1 **METRIC**

PROJECT 11-1111-0077
W.P. 2004-08-00
LOCATION N 4954845.0 ;E 279141.3
ORIGINATED BY JC
DIST Midland HWY 12
BOREHOLE TYPE 108mm I.D. Continuous Flight Hollow Stem Auger
COMPILED BY CS
DATUM Geodetic
DATE July 28, 2011
CHECKED BY KJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH kPa	WATER CONTENT	GR	SA	SI	CL
197.5	GROUND SURFACE									
0.0	ASPHALT									
0.1	Sand, trace to some gravel, trace to some silt (FILL) Compact Brown to grey Moist	1	SS	22						
		2	SS	13						
194.9 / 2.6	Sand and gravel, trace silt, trace clay (FILL) Compact Brown Moist	3	SS	13			35	60	4	1
193.8 / 3.7	SILTY CLAY to CLAY, trace sand Firm to very stiff Brown Moist	4	SS	2						
		5	SS	2	+ 1.7; + 3.4; + >100kPa; + >100kPa					
191.4 / 6.1	Silty SAND, some gravel, trace to some clay (TILL) Very dense Grey Wet	6	SS	100/0.03			13	52	27	8
		7	SS	100/0.08						
190.4 / 7.1	END OF BOREHOLE (AUGER/SPOON REFUSAL)									

NOTES:

1. Water level in open borehole at a depth 3.5 m (Elev. 194.0 m) upon completion of drilling.

2. Backfilled existing borehole, moved 3 m West and drilled unsampled new borehole to carry out vane tests at depths of 4.9 m (El. 192.6 m) and 5.2 m (El. 192.3 m) below ground surface. Installed piezometer with flush mount casing.

3. Water level in piezometer at a depth of 3.1 m (Elev. 194.4 m) on September 30, 2011.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%)

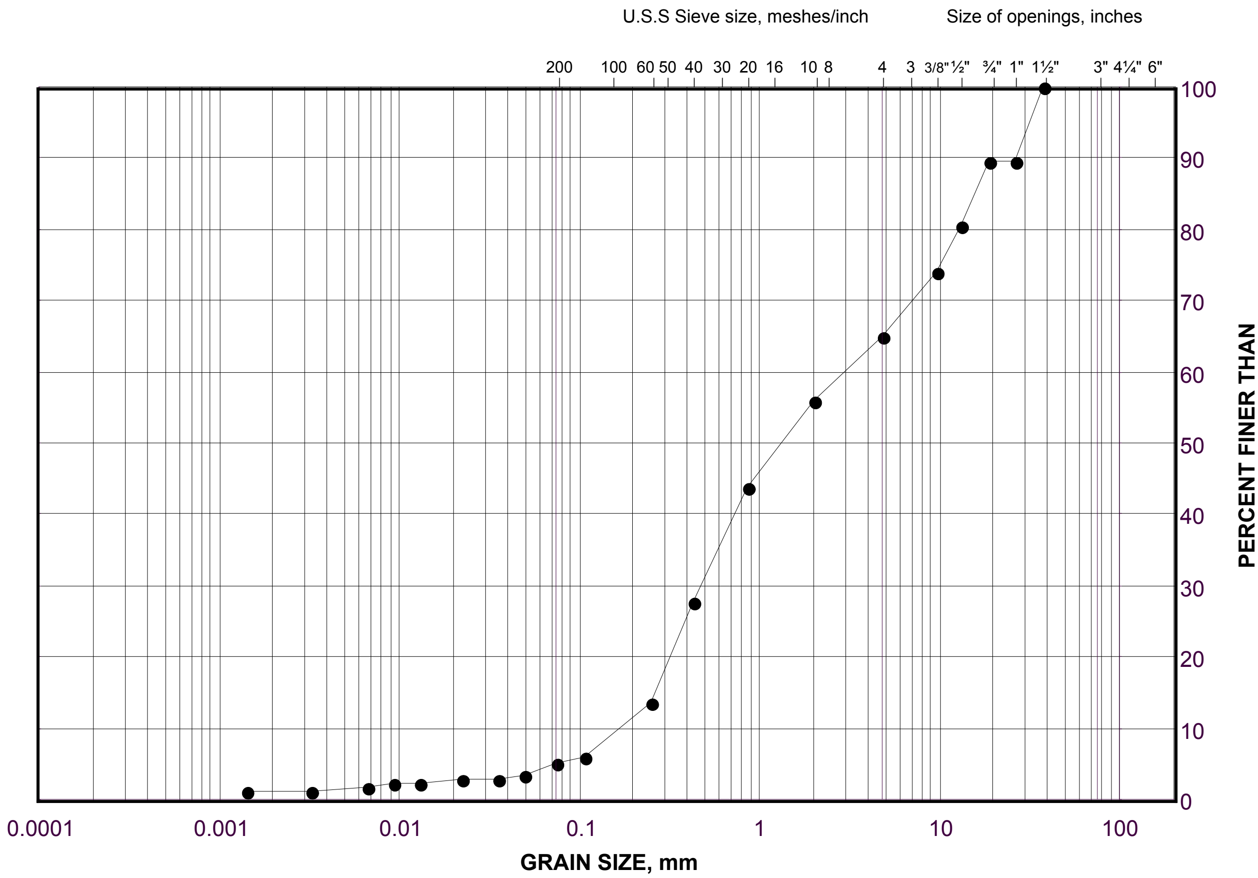
+ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

APPENDIX B

Laboratory Test Results, Current Investigation

GRAIN SIZE DISTRIBUTION
SAND and GRAVEL (Fill)

FIGURE B1



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

LEGEND

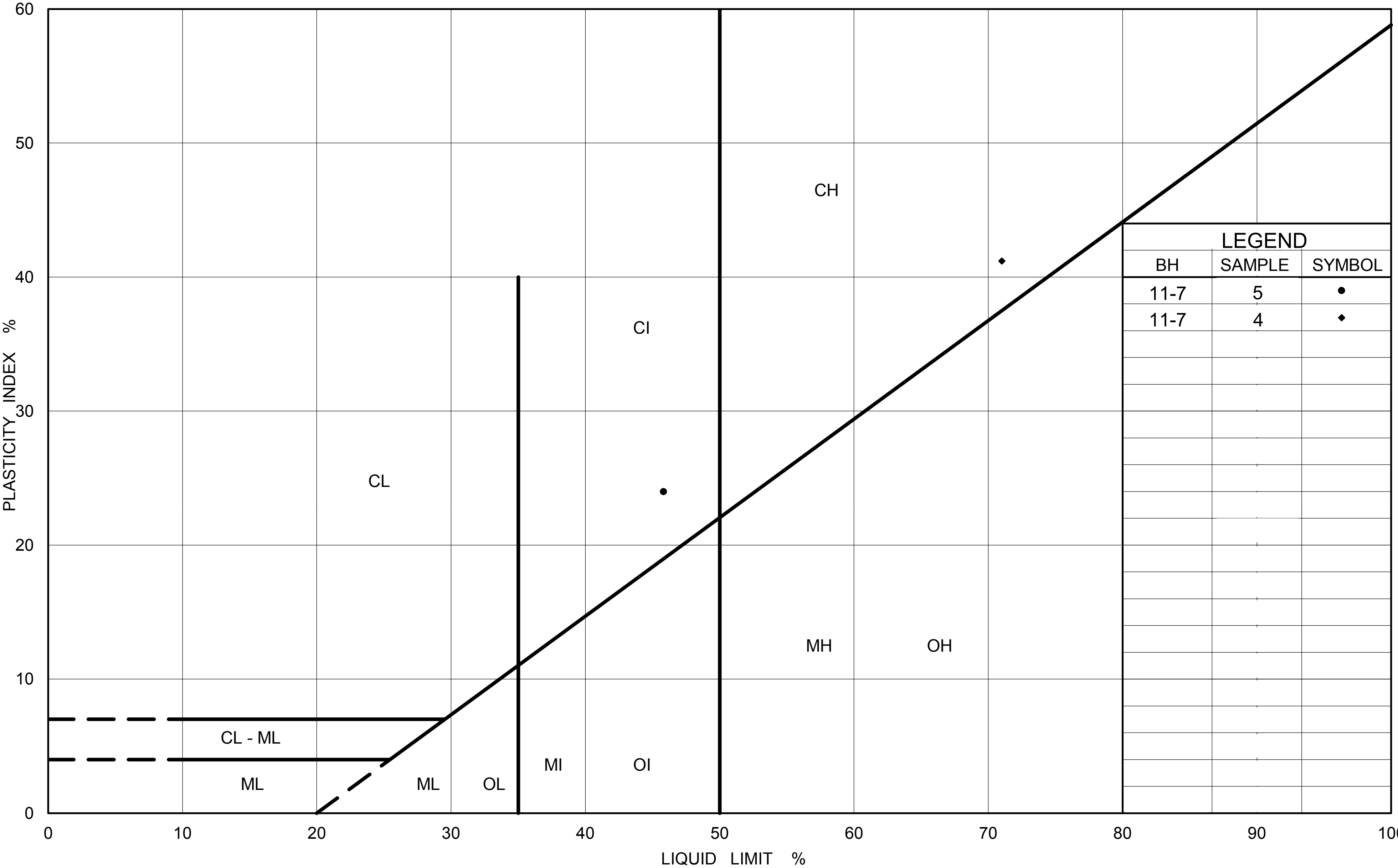
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	11-7	3	194.0

Project Number: 11-1111-0077B

Checked By: KJB

Golder Associates

Date: 20-Mar-12



BH	SAMPLE	SYMBOL
11-7	5	●
11-7	4	◆

Ministry of Transportation

Ontario

PLASTICITY CHART
SILTY CLAY TO CLAY

Figure No. B2

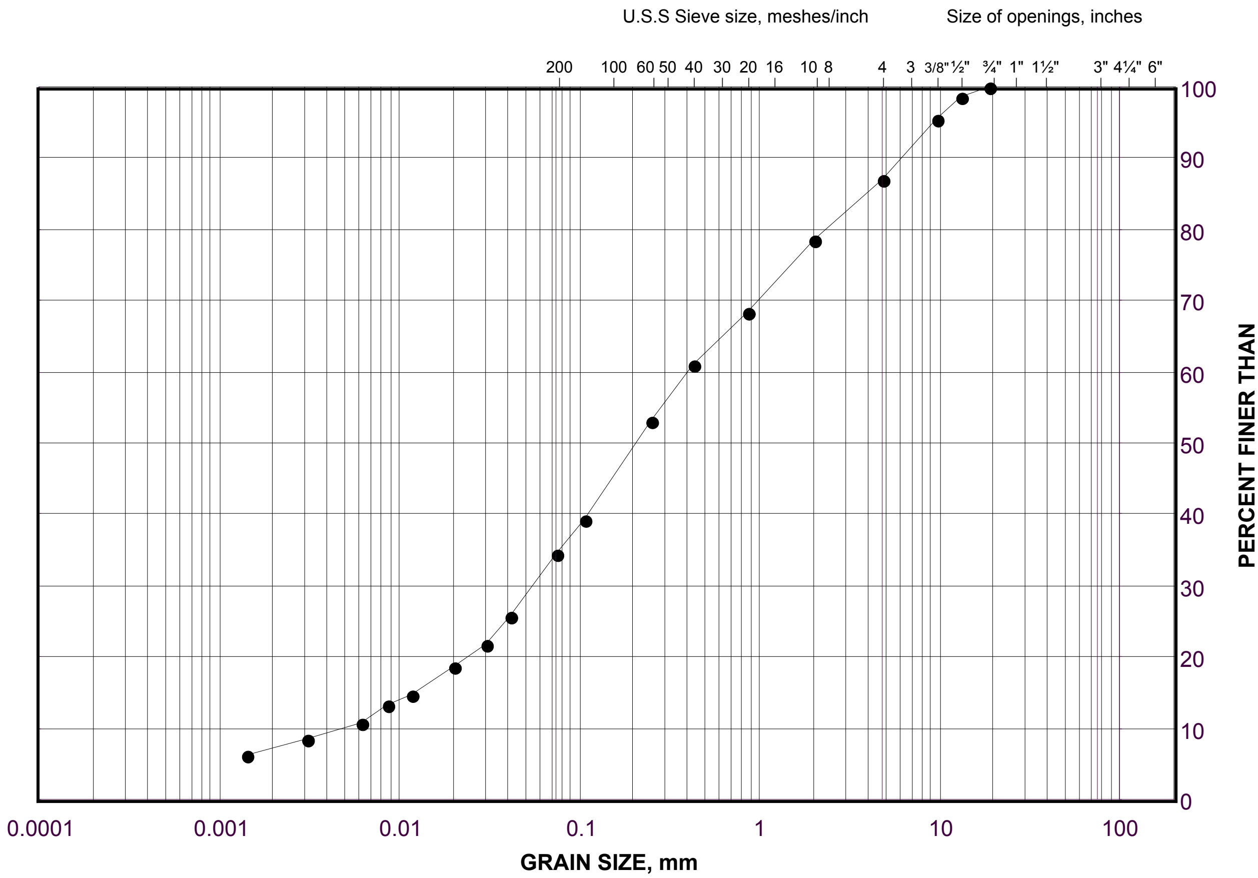
Project No. 11-1111-0077B

Checked By: KJB

GRAIN SIZE DISTRIBUTION

SILTY SAND (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)
●	11-7	6	191.0

Project Number: 11-1111-0077B

Checked By: KJB

Golder Associates

Date: 20-Mar-12

APPENDIX C

Record of Boreholes and Laboratory Test Results, Previous Investigation

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$ RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION — IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3·1416
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

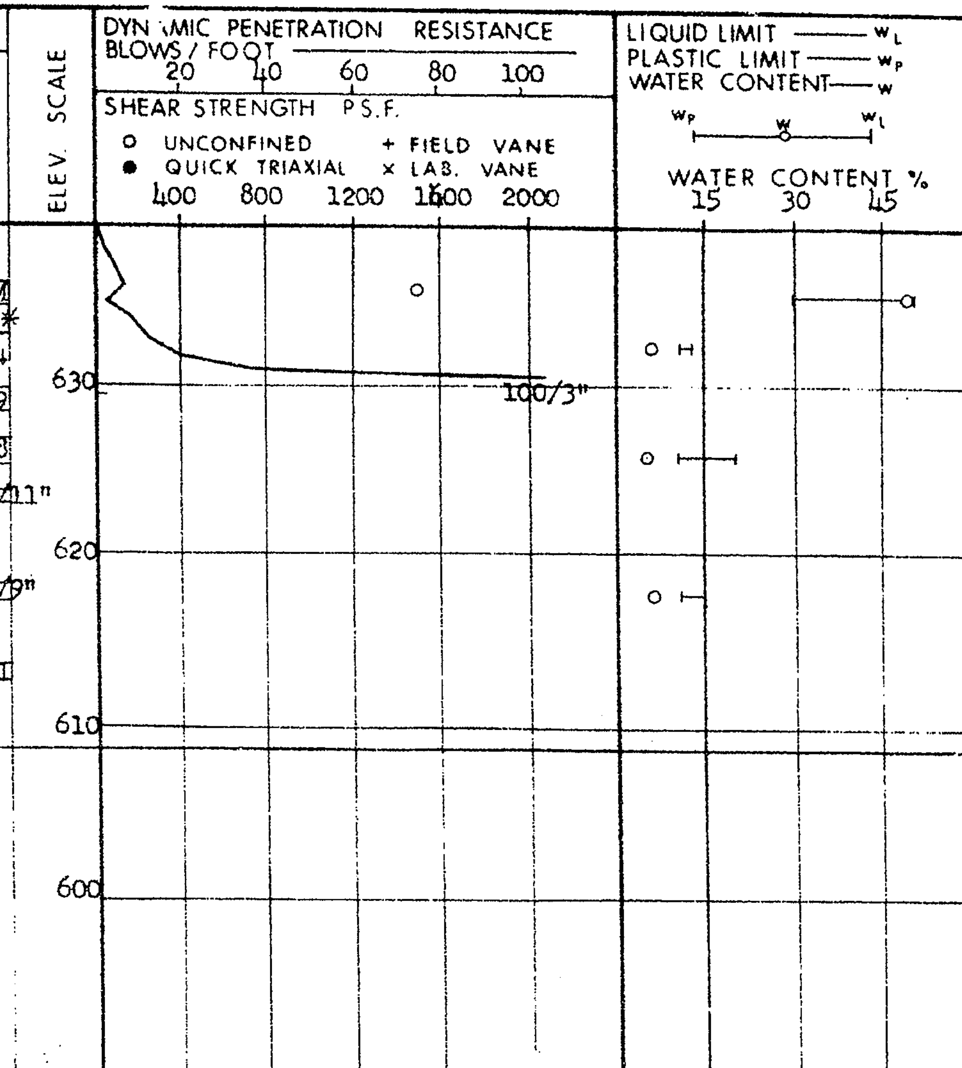
FOUNDATION SECTION

JOB 71-11029 LOCATION Sta. 580 + 40 ¢ Hwy. 12 Revn. Line 'L' o/s 20' Rt. ORIGINATED BY WH

W.P. 650-64-05 BORING DATE April 15-16, 1971 COMPILED BY WA

DATUM Geodetic BOREHOLE TYPE Diamond Drill, Washboring, NX Casing; Cone CHECKED BY [illegible]

	ELEV. DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS (GR, SA, SI, CL)
194.74	638.9	Ground Level					Art. Head ∇ (641) (195.4)
	0.0	Topsoil					
	1.0	Clayey silt to silty clay					▼ 637. (194.16)
193.37	634.4		1	TW	PM	107.5	
1.37	4.5	Glacial Till	2	SS	86 *		
			3	SS	24		21 41 30 8
		Compact to Dense	4	SS	49		* [illegible] boulders
191.08	626.9						
3.66	12.0		5	SS	158		
		Het. mix. of silt, sand and gravel, trace of clay	6	SS	136/11"		
		occ. boulders up to 18" in size.	7	SS	101/9"		28 32 32 8
		Very Dense	8	SS	161		Art. Head encountered ∇
185.59	608.9	Grey					
9.14	30.0	End of Borehole					



FORM OB-M[illegible]126 (REV. 1969) — OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1A

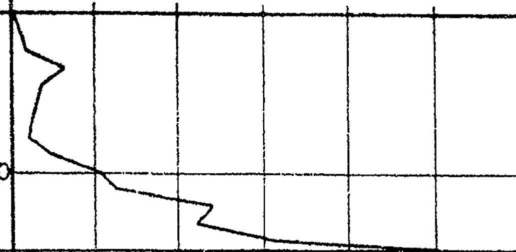
FOUNDATION SECTION

JOB 71-11029 — LOCATION Sta. 580 + 84 Ø Hwy. 12 Revn. Line'L' — ORIGINATED BY WH

W.P. 650-64-05 — BORING DATE April 16, 1971 — COMPILED BY WH

DATUM Geodetic — BOREHOLE TYPE Diamond Drill — CHECKED BY [illegible]

	SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (20 40 60 80 100); SHEAR STRENGTH P.S.F. (O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE)	LIQUID LIMIT — w_L, PLASTIC LIMIT — w_P, WATER CONTENT — w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
	ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
194.77	639.0	Ground Level									
0.0	0.0										
							630				
190.71	625.7										
4.05	13.3	End of Cone Test									
							620				



FORM OB-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

FOUNDATION SECTION

JOB 71-11029 | LOCATION Sta. 581 + 07 ℄ Hwy. 12 Revn. Line "L" o/s 17' Lt. | ORIGINATED BY WH

W.P. 650-64-05 | BORING DATE April 16 - 19, 1971 | COMPILED BY WH

DATUM Geodetic | BOREHOLE TYPE Diamond Drill - Washboring | CHECKED BY

	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
194.74	638.9	Ground Level					▼ 636.4 (193.97)
	0.0	Topsoil					
	1.0	Uniform Sand					
193.82 .03	635.9	Loose Brown					
.091	3.0	Clayey silt to silty clay. Firm. Greyish-Brown	1	SS	6		
192.82	632.6		2	TW	PM		
192	6.3	Glacial Till. Compact	3	SS	21	630	
		Limestone Boulder	3A	BX	86%		
		Het. mix. of silt, sand and gravel, trace of clay.	4	SS	112		26 39 30 5
			5	SS	100/5"		
		Very Dense				620	
188.49	618.4	Grey	6	SS	109/6"		13 40 37 10
6.25	20.5	End of Borehole				610	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE — 400 800 1200 1600 2000

+ > 2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w

WATER CONTENT %: 15 30 45

BULK DENSITY γ P.C.F.

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 71-11029 LOCATION Sta. 580 + 19 ¢ Hwy.12 Revn. Line 'L' o/s 20' Lt. ORIGINATED BY WH

W.P. 650-64-05 BORING DATE April 20-21, 1971 COMPILED BY WH

DATUM Geodetic BOREHOLE TYPE Diamond Drill - Washboring CHECKED BY

196.6

197 m

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000

LIQUID LIMIT — W_L; PLASTIC LIMIT — W_P; WATER CONTENT — W; WATER CONTENT % 15 30 45

BULK DENSITY γ P.C.F.

	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR SA SI CL
189.86	622.9	Ground Level					∇ 645.+ Art. Head
	0.0	Topsoil & Fill Material					∇ 621.9 (189.56)
0.46	1.5	Glacial Till occ. clayey silt seams up to 1" thick. Soft to Very Loose				620	
188.18	617.4		1	SS	3		18 37 31 14
1.68	5.5		2	SS	20		
			3	SS	130		
		Het. mix. of silt, sand and gravel, trace of clay	4	SS	80		
			5	SS	81	610	
		occ. boulders up to 4" in size.	6	SS	95		
			7	SS	157/10"	600	(181.7)
		Very Dense	8	SS	132		∇ 596. Art. Head encountered
180.26	591.4	Grey	9	SS	24		
9.6	31.5	End of Borehole				590	

FORM OB-MT 26 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 71-11029
W.P. 650-64-05
DATUM Geodetic

LOCATION Sta. 579 + 75 ℄ Hwy. 12 Revn. Line 'L' o/s 20' Rt.
BORING DATE April 23 - 26, 1971
BOREHOLE TYPE Washboring, NX Casing and Cone

ORIGINATED BY WH
COMPILED BY WA
CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH P.S.F.: o UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB. VANE: 400 800 1200 1600 2000
LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w
WATER CONTENT %: 15 30 45
BULK DENSITY γ P.C.F.

	ELEV. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
189.92	623.1	Ground Level					
0.0	0.0	Glacial Till occ. zones of clayey silt. Firm or Loose	1	SS	5	620	620.1 (189.00)
			2	SS	7		
188.27 / 1.68	617.7 / 5.5	Het.mix.of silt, sand & gravel, trace of clay	3	SS	88		22 39 28 11
			4	SS	155/10"		
		occ. boulders up to 5" in size.	5	SS	102	610	19 42 29 10
			6	SS	116		31 37 23 9
		Very Dense Grey	7	SS	85		
183.37 / 6.55	601.6 / 21.5	End of Borehole				600	
						590	

140/12"

FORM OB-MT-·26 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 71-11029
W.P. 650-64-05
DATUM Geodetic

LOCATION Sta. 579 + 60 ∅ Hwy. 12 revn. Line 'L' o/s 20' Lt.
BORING DATE April 27, 1971
BOREHOLE TYPE Washboring, NX Casing & Cone

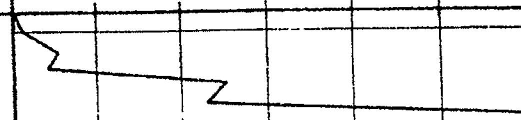
ORIGINATED BY WH
COMPILED BY WA
CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE; 400 800 1200 1600 2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %: 15 30 45

BULK DENSITY γ P.C.F.



	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS (GR SA SI CL)
189.22	620.8	Ground Level					⊽ 620. (188.98)
0	0.0	Topsoil & Fill Material				620	
188.37	618.0	Sand with grav. Compact	1	SS	33		
0.85	2.8	Glacial Till	2	SS	105/6"		
		Het.mix.of silt,sand and gravel,trace of clay	3	SS	130/9"		14 42 34 10
			4	SS	65/4"	610	
		occ. boulders up to 8" in size	5	SS	141		
			6	SS	50/1"		
		Very Dense	7	RC	79%		
182.64	599.2	Grey	8	SS	136	600	17 43 31 9
6.58	21.6	End of Borehole					

590

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

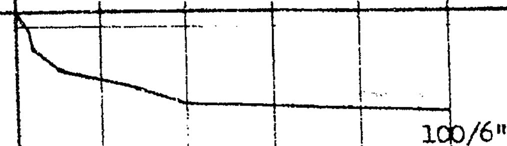
FOUNDATION SECTION

JOB 71-11029 LOCATION Sta. 579 + 10 ⊄ Hwy.12 Revn. Line 'L' o/s 20' Rt. ORIGINATED BY WH

W.P. 650-64-05 BORING DATE April 28 - May 4, 1971 COMPILED BY WA

DATUM Geodetic BOREHOLE TYPE Diamond Drill, washboring, NX, HX Casing, Cone CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000
LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT % 15 30 45
BULK DENSITY γ P.C.F.



	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	REMARKS (189.8m Art Head) GR. SA. SI. CL.
189.22	620.8	Ground Level				620	▼619. (189.0m) 188.7
	0.0	Topsoil & Fill Material					
188.46	618.3	Sa. with grav. Brown. Loose	1	SS	24		
0.76	2.5	Occ. silt seams up to	2	SS	54		31 57 (12)
187.54	615.3	½" thick	3	SS	165/9"		
1.68	5.5	Glacial Till	4	SS	200/7"	610	27 39 29 5
		Het. mix. of silt, sand and gravel, trace of clay	5	SS	154/11"		17 54 (29)
			6	SS	159/9"		
			7	SS	195/9"		
			8	RC	54%		
		Boulders up to 6" in size throughout	9	RC	33%		
			10	SS	210/8"	600	20 40 30 10
			11	RC	50%		
			12	RC	67%		
			13	SS	169/8"		
			14	BX	14%		
			15	SS	175/10"	590	
		Very Dense	16	BX	10%		
			17	SS	165		
			18	BX	8%		
176.72	579.8	Grey	19	RC	25%	580	∇ Artesian Water Encountered
12.5	41.0	Limestone Bedrock	20	RC	100%		
175.20	574.8	Sound Grey	21	BX	100%		
14.0	46.0	End of Borehole					

FORM OB-MT-[illegible]26 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 7

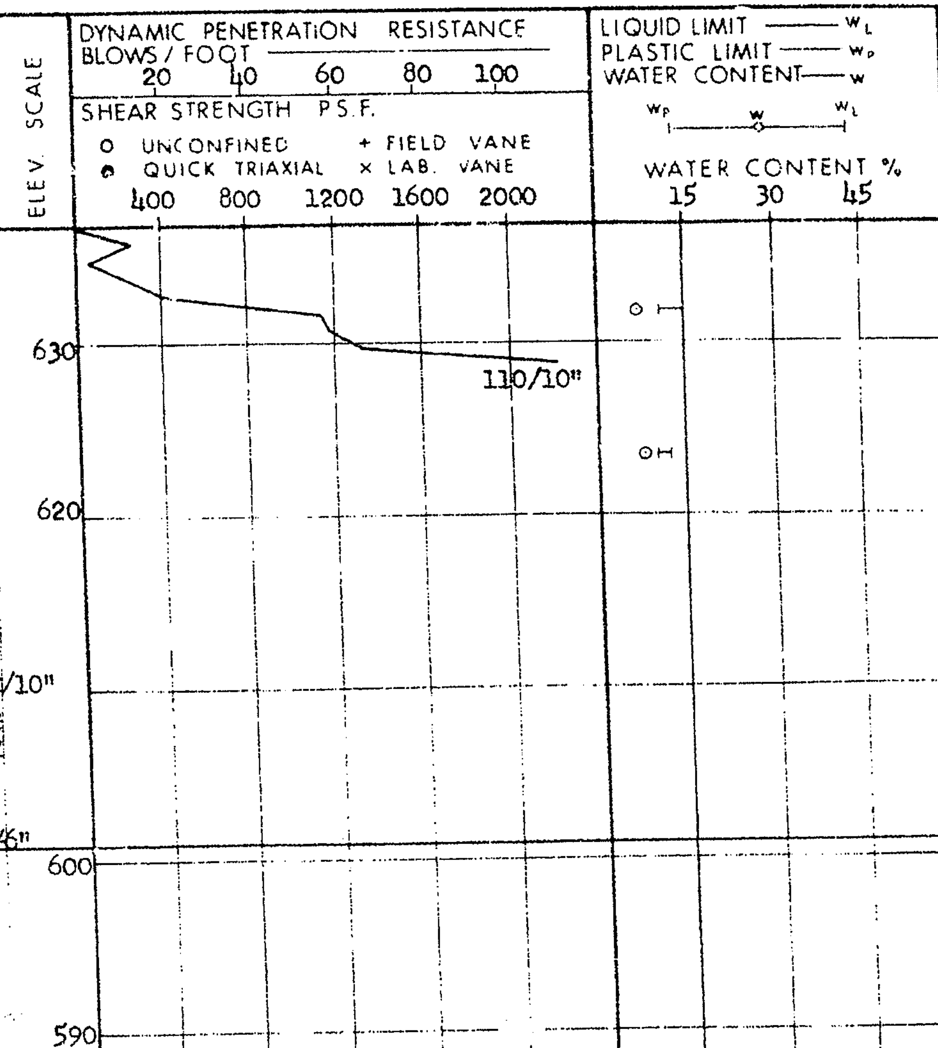
FOUNDATION SECTION

JOB 71-11029
W.P. 650-64-05
DATUM Geodetic

LOCATION Sta. 578 + 49 ⊄ Hwy.12 Rāvn. Line 'L' o/s 20' Rt.
BORING DATE May 4 - 6, 1971
BOREHOLE TYPE Washboring, NX Casing & Cone

ORIGINATED BY W H
COMPILED BY WH
CHECKED BY [illegible]

SOIL PROFILE		SAMPLES		
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT
194.04 636.6	Ground Level			
193.9 0 0.0	Topsoil			
193.27 634.1 0.76 2.5	Uniform sand. Compact. Brown	1	SS	21
	Glacial Till	2	SS	54
		3	SS	107
	Het.mix.of silt,sand and gravel, trace of clay	4	SS	20
		5	SS	27
		6	SS	72
	occ. boulders up to 4" in size	7	SS	85
		8	SS	102
		9	RC	19%
	Compact to Very Dense	10	SS	139/10"
		11	RC	15%
		12	RC	100%
		13	BX	8%
183.22 601.1		14	SS	100/6"
10.82 35.5	End of Borehole			



BULK DENSITY γ P.C.F

REMARKS GR.SA.SI.CL.
▼ 632. (192.63)
22 37 31 10
22 45 24 9

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 8

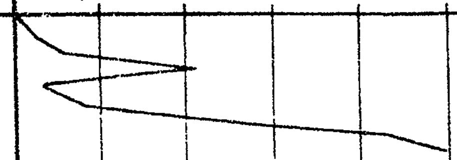
FOUNDATION SECTION

JOB 71-11029	LOCATION Sta. 578 + 95 ∅= Hwy.12 Revn. Line 'L' o/s 20' Lt.	ORIGINATED BY WH
W.P. 650-64-05	BORING DATE May 6-7, 1971	COMPILED BY WH
DATUM Geodetic	BOREHOLE TYPE Diamond Drill-Washboring	CHECKED BY

	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL.
194.77	639.0	Ground Level					
194.16 / 0.6	637.0 / 2.0	Fill Material Sand and Gravel / Clayey silt	1	SS	11		0 34 44 22
193.24 / 1.52	634.0 / 5.0	Firm to Stiff. Brown	2	SS	60		23 39 33 7
		Glacial Till	3	SS	146		▼ 633. (192.94)
		Het.mix.silt,sand and gravel. Trace of clay.				630	
			4	SS	150/8"		22 44 27 7
190.96 / 3.81	626.5 / 12.5	Silty Sand. Dense	5	BX	40%		
			6	SS	41		
4.27	14.0		7	BX	35%		
			8	SS	70		
		Very Dense	9	BX	19%	620	
188.15	617,3	Grey	10	SS	185		
			11	SS	162/8"		
6.61	21.7	End of Borehole					
						610	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB. VANE 400 800 1200 1600 2000



LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w

WATER CONTENT % 15 30 45

BULK DENSITY γ P.C.F.

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

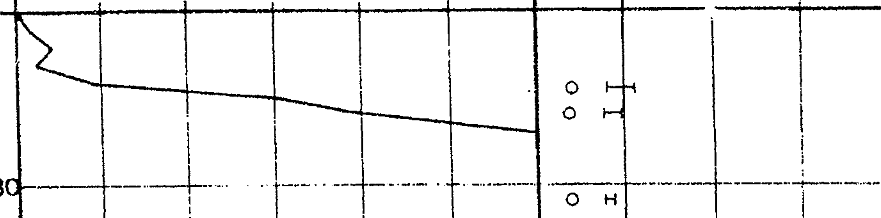
RECORD OF BOREHOLE No. 9

FOUNDATION SECTION

JOB 71-11029	LOCATION Sta. 578 + 98 Ø Hwy.12 Revn. Line 'L' o/s 130' Rt.	ORIGINATED BY WH
W.P. 650-64-05	BORING DATE May 10-11, 1971	COMPILED BY WH
DATUM Geodetic	BOREHOLE TYPE Diamond Drill-Washboring	CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR. SA. SI. CL.
195.07 640.0						
0.0	Clayey silt with sand					
194.16 637.0	Very Stiff. Brown	1	SS	18		21 38 31 10
0.91 3.0	Glacial Till	2	SS	59		19 40 31 10
	Het.mix.of silt,sand &	3	SS	93		
	gravel, trace of clay.	4	SS	30	630	35 33 27 5
		5	SS	52		
	occ. boulders up to	6	SS	58		
	4" in size	7	SS	143		21 40 30 9 Hole dry to ∓621.(189.28)
	Dense to Very Dense	8	SS	150/6"	620	Cave at 621.
	Grey	9	BX	25%		24 53 (23)
187.15 614.0		10	SS	2",08*		
7.92 26.0	End of Borehole				610	*loss of Drill water

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB. VANE 400 800 1200 1600 2000



LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w
WATER CONTENT % 15 30 45

BULK DENSITY γ P.C.F.

FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 10

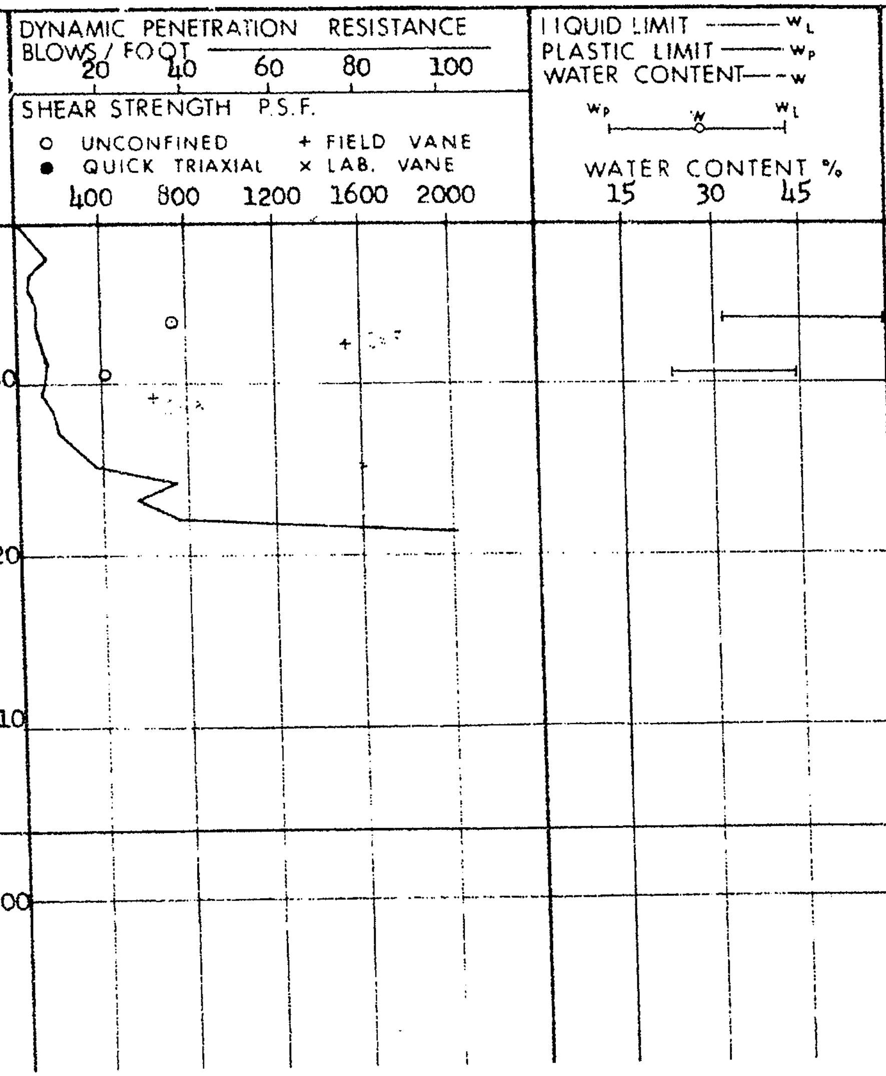
FOUNDATION SECTION

JOB 71-11029 LOCATION Sta. 582 + 18 ℄ Hwy. 12 Rev'n. Line 'L' o/s 109' Lt. ORIGINATED BY WH

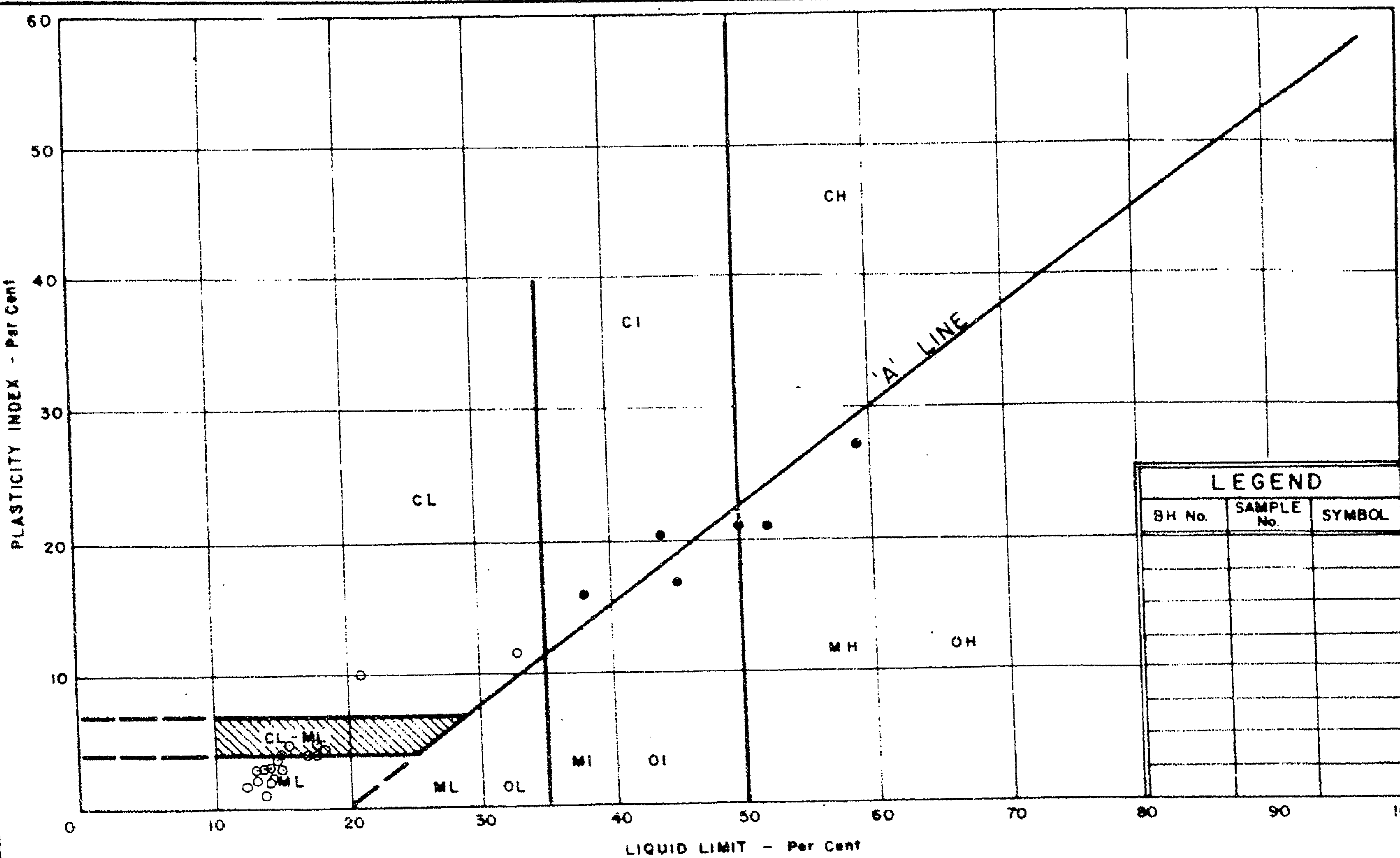
W.P. 650-64-05 BORING DATE May 12 - 13, 1971 COMPILED BY WH

DATUM Geodetic BOREHOLE TYPE Diamond Drill - Washboring CHECKED BY

	ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
194.77	639.0	Ground Level						
194.00	0.0	Topsoil						
	636.5	Sand - Loose - Brown	1	SS	7			635. (193.55)
0.76	2.5	Silty clay to clay	2	TW	PM		99	
		Firm	3	TW	PM	630	105	
191.26	627.5	Brownish Grey						
3.51	11.5	Glacial Till	4	SS	6			18 50 22 10
		Saturated	5	SS	9			
189.43	621.5	Loose to Compact	6	SS	12			
5.33	17.5		7	SS	53	620		35 46 (19)
		Het.mix.of silt,sand & gravel, trace of clay	8	SS	160/9"			
			9	BX	37%			
			10	SS	130/6"			
		occ. boulders up to 10" in size.	11	BX	28%	610		67 26 (7)
			12	SS	135/9"			
184.10	604.0	Very Dense Grey	13	BX	30%			
10.67	35.0	End of Borehole				600		



FORM OB-MT-[illegible]52
REVISED JUL[illegible] 1966



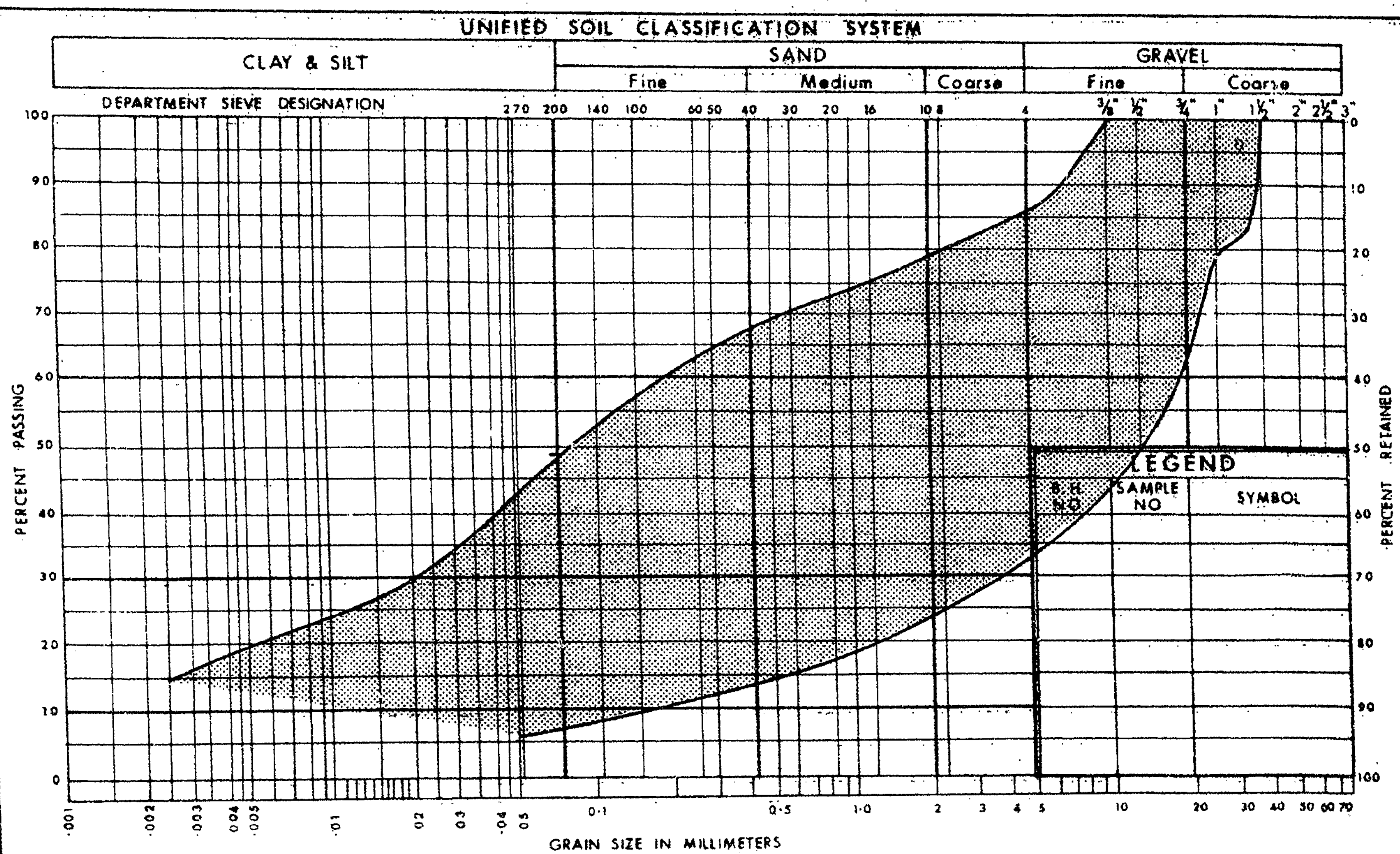
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART

● — CLAYEY SILT TO SILTY CLAY
○ - GLACIAL TILL

W.P. No.	650-64-05
JOB No.	71-11029
FIG. 1	

FORM OE MT-17
REV. APR. 1965



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION

GLACIAL TILL

W.P. No. 650-64-05

JOB No: 71-11029

FIG. 2

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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