



July 2010

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN CARP ROAD UNDERPASS REHABILITATION AND WIDENING STRUCTURE SITE 3-287 HIGHWAY 417 EXPANSION FROM EAGLESON ROAD TO HIGHWAY 7 G.W.P. 255-98-00

Submitted to:

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REPORT



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Table of Contents

PART A - FOUNDATION INVESTIGATION REPORT

| | | |
|-------|--------------------------------------|---|
| 1.0 | INTRODUCTION | 1 |
| 2.0 | SITE DESCRIPTION | 2 |
| 3.0 | INVESTIGATION PROCEDURES | 3 |
| 4.0 | SITE GEOLOGY AND STRATIGRAPHY | 5 |
| 4.1 | Regional Geological Conditions | 5 |
| 4.2 | Site Stratigraphy | 5 |
| 4.2.1 | Fill Material and Topsoil | 6 |
| 4.2.2 | Sand | 6 |
| 4.2.3 | Till | 7 |
| 4.2.4 | Refusal and Bedrock | 7 |
| 4.2.5 | Groundwater Conditions | 8 |
| 5.0 | CLOSURE | 9 |

PART B - FOUNDATION DESIGN REPORT

| | | |
|-------|------------------------------------------------------------|----|
| 6.0 | DISCUSSION AND ENGINEERING RECOMMENDATIONS | 10 |
| 6.1 | General | 10 |
| 6.2 | Assessment of Existing Abutment and Pier Foundations | 10 |
| 6.2.1 | Founding Elevations | 10 |
| 6.2.2 | Geotechnical Resistance | 11 |
| 6.2.3 | Resistance to Lateral Loads | 12 |
| 6.3 | Seismic Site Coefficient and Liquefaction Assessment | 12 |
| 6.4 | Lateral Earth Pressures for Design | 13 |
| 6.5 | Approach Embankment Design and Construction | 15 |
| 6.5.1 | Subgrade Preparation and Embankment Construction | 16 |
| 6.5.2 | Approach Embankment Stability | 16 |
| 6.6 | Construction Considerations | 17 |
| 6.6.1 | Excavations and Temporary Excavation Support | 17 |
| 6.6.2 | Groundwater and Surface Water Control | 17 |
| 7.0 | CLOSURE | 18 |



Drawing 1
Figure 1
Appendix A
Appendix B

DRAWINGS

DRAWING 1 - Borehole Locations and Soil Strata

FIGURES

FIGURE 1 - Grain Size Distribution - Sand

APPENDICES

APPENDIX A

List of Abbreviations and Symbols
Records of Borehole Sheets 07-401 and 07-402

APPENDIX B

Record of Borehole Sheets, 1971 Investigation by MTO
Grain Size Distributions from 1971 Investigation by MTO



PART A
FOUNDATION INVESTIGATION
CARP ROAD UNDERPASS REHABILITATION AND WIDENING
STRUCTURE SITE 3-287
HIGHWAY 417 EXPANSION
FROM EAGLESON ROAD TO HIGHWAY 7
G.W.P. 255-98-00



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation associated with the expansion of Highway 417 from Eagleson Road westerly to Highway 7, in Ottawa, Ontario.

Foundation investigation services are required on this project for the following components:

- High Fill embankment widening and structure modifications at Carp Road;
- Replacements at Culvert 60 and 60D2 near Eagleson Road;
- New High Mast Lighting poles and overhead signs;
- High Fill embankments for the realigned ramps at Carp Road; and,
- Carp River bridge widening and replacement.

This report addresses the high fill embankment widening and structure modifications of the existing Carp Road underpass structure crossing over Highway 417.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposals (RFP) dated February 6, 2007. The work was carried out in accordance with Golder's Quality Control Plan dated November 2007.



2.0 SITE DESCRIPTION

The existing underpass is located at the intersection of Carp Road and Highway 417 in Ottawa, Ontario. The underpass is aligned approximately northwest to southeast and crosses Highway 417 approximately perpendicularly. The underpass carries local traffic along Carp Road over the westbound and eastbound lanes of Highway 417. Below the underpass, Highway 417 consists of a four-lane median-divided highway.

A municipal landfill is located to the west of the structure, an aggregate pit is located north of the structure, a retail fuel outlet is located south of the structure, and a construction contractor storage yard and snow dump area are located east of the structure. Recently high mast lighting has been installed at the northeast, southeast, and southwest quadrants of the interchange, within the existing access ramps, as well as in the center median along Highway 417.

The existing underpass structure is a two span reinforced concrete bridge supported on two center pier columns and on abutment walls. Both spans are approximately 38 m long. The bridge also has 7.5 m long concrete approach slabs behind each abutment. The existing underpass is tapered in width, with two lanes (total of 13.9 m wide) at the northern abutment, widening to three lanes (total of 15.7 m wide) at the southern abutment. The approach embankments have sideslopes inclined at about 4H:1V and foreslopes inclined at about 2H:1V. The sideslopes appeared to be stable, with well established vegetation, at the time of our fieldwork. The foreslopes are covered with slope paving.

The underpass was constructed in the early 1970's. The results of the foundation investigation are provided in MTO's GEOCRE No. 31G5-102, W.P. 437-64-00, Hwy. 417, Regional Road 5 Crossing. Subsequent to the bridge construction, in the early 1980's a retaining wall (Structure No. 3-287) was constructed under the underpass structure at the toe of the north approach embankment foreslope along Highway 417, to allow for additional roadway width. This work was carried out under W.P. 5-80-01 / Contract No. 81-51.



3.0 INVESTIGATION PROCEDURES

A foundation investigation was carried out for the existing underpass in late April of 1971 and the results of that investigation are summarized in MTO's GEOCREs No. 31G5-102, Foundation Investigation Report W.P. 437-64-00, Hwy. 417, Regional Road 5 Crossing. This investigation consisted of ten boreholes with Standard Penetration Testing (SPT) using a split spoon sampler and dynamic cone penetration testing. The laboratory testing consisted of a limited number of grain size distribution tests and water content determinations. Of the ten boreholes put down, six boreholes were located near the foundations and four boreholes were located along Highway 417. Overall, seven of the ten previous boreholes are located within the current project area.

The current subsurface investigation was carried out at the approach embankments of the existing underpass. On November 13 and 14, 2007, two boreholes (numbered 07-401 and 07-402) were advanced at the locations shown on Drawing 1. The boreholes were put down approximately 20 m behind each abutment and along the western slope of each approach embankment.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers on a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to practical auger refusal at depths of 9.9 and 10.2 m below the existing ground surface (Elevations 116.6 m and 117.0 m) at Boreholes 07-401 and 07-402, respectively.

Soil samples were obtained at 0.75 metre intervals using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with SPT procedures.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work.

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination. Classification tests consisting of grain size distribution tests were carried out on selected soil samples at the Ottawa laboratory. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The borehole locations were selected and positioned by Golder relative to existing site features. The borehole elevations and locations were subsequently determined by J.D. Barnes Ltd. Land Surveyors. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum are summarized in the following table and are shown on Drawing 1.

| Borehole Number | Borehole General Location | MTM NAD83 Northing (m) | MTM NAD83 Easting (m) | Ground Surface Elevation (m) |
|------------------------|------------------------------------------------------------|-------------------------------|------------------------------|-------------------------------------|
| 07-401 | Carp Road, west shoulder of south approach embankment west | 5015771.0 | 347417.0 | 126.5 |
| 07-402 | Carp Road, west shoulder of north approach embankment | 5015860.2 | 347342.6 | 127.2 |
| 1 | North Abutment, NE corner | 5015856.2 | 347371.6 | 130.2 |



CARP ROAD UNDERPASS REHABILITATION AND WIDENING

| Borehole Number | Borehole General Location | MTM NAD83 Northing (m) | MTM NAD83 Easting (m) | Ground Surface Elevation (m) |
|-----------------|---------------------------------|------------------------|-----------------------|------------------------------|
| 2 | North Abutment, NW corner | 5015848.5 | 347363.1 | 130.1 |
| 3 | Center Pier, east side | 5015829.5 | 347395.6 | 130.5 |
| 4 | Center Pier, west side | 5015821.8 | 347386.9 | 130.5 |
| 5 | South Abutment, SE corner | 5015802.7 | 347419.9 | 130.7 |
| 6 | South Abutment, SW corner | 5015794.9 | 347411.3 | 130.6 |
| 10 | Hwy 417 west bound travel lanes | 5015835.3 | 347374.8 | 130.4 |

The MTM coordinates for the boreholes (1 through 6 & 10) put down in the 1971 subsurface investigation were interpreted from base plans showing the original Carp Road stationing.



4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the Smiths Falls Limestone Plain minor physiographic region, as delineated in *The Physiography of Southern Ontario*¹ that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

Most of the Ottawa-St. Lawrence Lowland physiographic region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones, and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield.

The Smiths Falls Limestone Plain is characterized by shallow overburden deposits overlying limestone bedrock of the Ottawa Formation; this formation consists of grey limestone with some shaly partings and seams². The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock.¹

4.2 Site Stratigraphy

For the subsurface investigation at this site, the existing GEOCRESS information was supplemented by two boreholes advanced along the embankments and 20 m behind the abutments of the existing Carp Road underpass structure. The borehole locations, ground surface elevations and an interpreted stratigraphic profile are shown on Drawing 1, including both the two new boreholes as well as seven boreholes from the MTO 1971 investigation.

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets (Appendix A) and on Figure 1. The Record of Borehole sheets from the 1971 MTO investigation are provided in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the abutments and approaches to the underpass structure consist of a thin (0.8 m thick) layer of discontinuous sandy fill material underlain by a thick deposit of native sands which extends to elevations ranging from 116.7 m to 121.9 m and has a thickness ranging from about 6 to 11 m. The sand deposit is underlain by a silty sand and gravel till deposit, which is up to approximately 5 m thick. At the center pier location, the surface of the till deposit is higher and it appears that the overlying native sand deposit was completely removed during the construction of Highway 417.

The bedrock surface and/or auger refusal were encountered between Elevations 116.5 m and 117.4 m. The bedrock consists of limestone with shale seams.

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

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.



A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill Material and Topsoil

Both of the boreholes put down for the current investigation encountered fill material at ground surface.

All of the boreholes advanced for the MTO 1971 investigation also encountered fill material at ground surface. However, due to construction of the underpass structure, the ground surface at the north and south abutments has been lowered by approximately 2.4 and 2.5 m, respectively. The ground surface along the traveled lanes of Highway 417 and at the center pier location has been lowered by as much as 9.2 metres. As a result of this grading, the fill materials encountered in the 1971 MTO boreholes have since been removed.

At Boreholes 07-401 and 07-402, located adjacent to the approach embankments, the upper 0.2 m of the embankment fill material consists of sandy topsoil fill. About 0.6 m of embankment fill material exists below the topsoil layer, consisting primarily of fine sand and fine to coarse sand.

In Borehole 07-401, the one SPT N value obtained in the fill material was 1 blow per 0.3 m of penetration, indicating it to be very loose.

4.2.2 Sand

Native sand deposits were encountered below the topsoil and fill material at all of the boreholes, with thicknesses relative to current ground surface levels ranging from about six to nine metres. This sand extends to Elevation 117.1 at Borehole 07-402 (north side) and gradually thins to the south (in conjunction with a rise in the surface of the underlying glacial till). At its thinnest location, the sand previously only extended down to about Elevation 123.0 m. However, due to construction of the highway at an elevation lower than that level, the sands have been removed by site grading from beneath the westbound travelled lanes and the center pier locations. The sand then thickens again to the south, extending down to Elevation 116.7 m at Borehole 07-401 (the most southerly borehole).

These sand deposits consist of stratified fine to coarse sands with varying amounts of gravel and fines. The results from the four gradation tests carried out on samples of this material from the current investigation are shown on Figure 1 while the results of three tests from the previous MTO investigation are provided in Appendix B.

The measured natural water contents ranged from 10 to 17 percent.

The SPT N values were in the range of 10 blows to 188 blows per 300 millimetres of penetration, averaging about 60. Gravels within the sands may have exaggerated some of the blow count values in the SPT. The SPT N values are also notably less in the two boreholes from the current investigation (N values of 10 to 38 blows per 300 millimetres of penetration) versus the measurements in the 1971 boreholes, which may reflect differences in testing equipment or procedures. The SPT N values from the current investigation indicate that the sand deposits are in a compact to dense state.



4.2.3 Till

The native sand deposits are generally underlain by glacial till. As described previously, the surface of the glacial till rises from north to south, 'peaking' in the area of the north abutment and centre pier, and then deepens again to the south. The glacial till was not however encountered in Boreholes 07-401 and 07-402, advanced behind the north and south abutments, where bedrock was encountered directly below the native sands.

This deposit was fully penetrated at MTO Boreholes 1, 4 and 5 at Elevations of about 117.4 m, 116.5 m, and 117.1 m, respectively. At MTO boreholes 2 and 3, the glacial till was proven to Elevations of about 118.0 and 119.3, respectively.

The glacial till is considered to be a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand. The results from the six gradation tests carried out on this material for the previous 1971 MTO investigation are provided in Appendix B. However these samples were retrieved using a 50 mm diameter sampler and therefore do not reflect the cobble and boulder portions of the deposit.

The measured natural water contents ranged from 6 to 13 percent.

SPT testing could only be carried out sporadically throughout the till stratum due to the presence of cobbles and boulders. Rotary diamond drilling techniques were at times required to penetrate the deposit. SPT N-values of the successful tests ranged from 82 blows per 300 millimetres of penetration to over 200 blows, which indicate a very dense state of packing. However gravel and cobbles within the till may have exaggerated some of the blow count values in the SPT.

4.2.4 Refusal and Bedrock

Practical refusal to augering was encountered at Borehole 07-401 at a depth of about 9.9 m (Elevation 116.6 m) and at Borehole 07-402 at a depth of about 10.2 m (Elevation 117.0 m).

Bedrock was confirmed at MTO Boreholes 1, 4, and 5, which were advanced beyond the refusal depth for an additional 2.3 m, 3.1 m, 3.1 m into the bedrock, respectively. The bedrock encountered consists of limestone with shale seams of varying thicknesses, being typically less than 50 mm.

The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations. It should be noted that the bedrock was cored in three of the boreholes; the surface of the limestone bedrock was inferred in the two remaining boreholes by refusal to advance the augers.

| Borehole Number | Existing Ground Surface Elevation (m) | Depth to Bedrock (m) | Bedrock Surface Elevation (m) |
|-----------------|---------------------------------------|----------------------|-------------------------------|
| 07-402 | 127.2 | 10.1 | 117.1 |
| 1 | 130.2* | 12.8 | 117.4 (cored) |
| 4 | 130.5* | 14.0 | 116.5 (cored) |
| 5 | 130.7* | 13.6 | 117.1 (cored) |
| 07-401 | 126.5 | 9.8 | 116.7 |

*Note: Existing ground surface elevation and depth to bedrock were established at the time the borehole was drilled in 1971.



4.2.5 Groundwater Conditions

Groundwater levels were observed in the open boreholes at the completion of each borehole. For the 1971 MTO boreholes, groundwater observations were taken several days after completion. The groundwater level in each borehole is summarized in the table below:

| Borehole Number | Existing Ground Surface Elevation (m) | Water Level Depth (m) | Water Level Elevation (m) | Date |
|-----------------|---------------------------------------|-----------------------|---------------------------|---------------|
| 07-402 | 127.2 | 9.1 | 118.1 | November 2007 |
| 1 | 130.2 | 11.0 | 119.2 | April 1971 |
| 2 | 130.1 | 10.9 | 119.2 | April 1971 |
| 3 | 130.5 | 9.8 | 120.7 | April 1971 |
| 4 | 130.5 | 10.4 | 120.1 | April 1971 |
| 5 | 130.7 | 10.0 | 120.7 | April 1971 |
| 6 | 130.6 | 10.2 | 120.4 | April 1971 |
| 07-401 | 126.5 | 7.0 | 119.5 | November 2007 |

*Note: Existing ground surface elevation and water level depth were established at the time the borehole was drilled in 1971.

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events. It should also be noted that the groundwater levels may have also been influenced by nearby quarry and landfill operations.



5.0 CLOSURE

This report was prepared by the Project Manager, Mr. Michael Cunningham P.Eng. This report was reviewed by Mr. Fintan J. Heffernan P.Eng, the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

Mike Cunningham, P.Eng.
Associate



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Designated MTO Contact



BDG/MIC/FJH/am/cg

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PART B
FOUNDATION DESIGN
CARP ROAD UNDERPASS REHABILITATION AND WIDENING
STRUCTURE SITE 3-287
HIGHWAY 417 EXPANSION
FROM EAGLESON ROAD TO HIGHWAY 7
G.W.P. 255-98-00



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the design and construction of the proposed rehabilitation and widening/modification of the existing Carp Road underpass structure associated with the expansion of Highway 417.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are only intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed rehabilitation/widening. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that, as part of the rehabilitation, the deck will be widened by approximately 84 mm to the east. It is also understood that both abutments will be changed to semi-integral configurations. The top of the associated wing walls will also be removed and the new approach slabs cantilevered out over the top of the existing wing walls. However the center pier will not be modified under this assignment.

6.2 Assessment of Existing Abutment and Pier Foundations

The structure modifications will alter the foundation loading and also require a reassessment of the existing foundations in relation to the current CHBDC. An assessment of the available resistances for those foundations in comparison to the proposed new design foundation loads has therefore been carried out.

As discussed below, it has been determined that the available bearing resistances for the existing foundations will be sufficient for the proposed construction and therefore retrofitting of the foundations will not be required.

6.2.1 Founding Elevations

Based on the available 1972 design drawings, the existing underpass structure is supported on spread footings that are founded on native soils. Based on the records of the 1971 boreholes, it is inferred that the existing abutment foundations are bearing on native sands, while the center pier footings are likely supported on the underlying glacial till. The following table presents the approximate footing dimensions and design founding elevation as indicated from the available 1972 design drawings for the existing underpass. The bedrock surface elevation for each foundation element is also indicated, based on interpolation of the 1971 borehole investigation results.

| Foundation Element | Design Footing Dimensions | Design Footing Founding Elevation | Bedrock Surface Elevation |
|--------------------|----------------------------------------|-----------------------------------|---------------------------|
| North Abutment | 3.1 m x 13.6 m x 0.6 m | 123.1 m | 117.1 m |
| Center Piers | 4.3 m x 4.3m x 1.2 m (two footings) | 119.6 m | 116.5 m |
| South Abutment | 3.1 m x 15.4 m x 0.6 m | 123.4 m | 117.1 m |



6.2.2 Geotechnical Resistance

The factored geotechnical resistance values at Ultimate Limit States (ULS) and the resistance values at Serviceability Limit States (SLS) have been assessed based on the indicated founding levels, footing dimensions, and bearing strata. For this analysis, the SLS bearing resistance is defined as that corresponding to a settlement less than 25 mm.

| Existing Foundation Element | ULS (kPa) | SLS (kPa) |
|-----------------------------|-----------|-----------|
| North Abutment | 500 | 370* |
| Center Pier Footings | 1,000 | 600 |
| South Abutment | 500 | 310* |

* SLS values are based on effective width of existing footing.

The SLS and factored ULS resistance values in the above table for the centre pier footings are significantly larger than those for the north and south abutment footings since the pier footings are supported on till while the abutment footings are supported on the sand deposit.

The actual bearing pressure values (for the existing foundation geometry but with the new design loads) are understood to be as follows:

| Abutments | | Centre Pier | |
|--------------------------|---------|-------------------|---------|
| Bearing Condition | Value | Bearing Condition | Value |
| Max. SLS (toe) | 391 kPa | Max SLS | 592 kPa |
| Min SLS (heel) | 39 kPa | Min SLS | 288 kPa |
| Average SLS | 215 kPa | Average SLS | 480 kPa |
| Effective SLS (Net Area) | 302 kPa | | |
| ULS | 374 kPa | ULS | 648 kPa |

The factored ULS resistance values exceed the design loading. However the SLS resistance values given in the above table are less than the 'peak' design toe pressures. It is considered however that, given the varying bearing pressures across the width of these footings, particularly for the abutment footings (due to the eccentric loading), the 'peak' pressures are not considered to provide a reasonable assessment of the settlement response of the footings. The footing settlements would more realistically depend on the overall stress distributions, not the 'peak' stresses, since the stress distribution impacts on the depth and width of the zone of soil that will be influenced by that loading. For that reason, the bearing resistance is in fact a function of the footing/bearing width. Using the 'peak' bearing SLS stress in conjunction with a bearing resistance determined using the full footing width (as though that peak stress acted over the full footing width) would provide an overly conservative assessment of the footing settlement response. The SLS resistance values given above for the abutment footings can therefore be assessed using the 'effective' width of the loaded area, based on the stress



distributions. It is considered that the 'effective' bearing stresses, acting over those 'effective' bearing widths, would provide a more reasonable assessment of the performance of these footings. For the centre pier, it is considered that the 'average' bearing pressure can be used (due to the much lesser variation in the bearing pressure across the footing width) and this pressure is similarly less than the SLS bearing resistance.

In summary, the resistance values for all of the existing footings are understood to be sufficient to resist the design loads, and no modifications to the footings should be required.

The above resistance values are dependent on the footing size and configuration; the geotechnical resistances should therefore be reassessed if the existing presumed footing widths or founding elevations differ from those given in Section 6.2.1.

The factored ULS geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

6.2.3 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and the native soils should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the concrete and the native soils may be taken as 0.55. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.3 Seismic Site Coefficient and Liquefaction Assessment

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.0, consistent with Soil Profile Type I.

Seismic liquefaction occurs when earthquake vibrations cause an increase in the pore water pressure within the soil, which reduces the effective stress between the soil particles and the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Large lateral movements of even gently sloping ground, referred to as "lateral spreading", which could impact on embankment stability;
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; and,
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements. The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);
- Soils having a loose state of packing; and,



- Soils located below the groundwater level.

The assessment of the potential seismic liquefaction hazard at this site involves comparing the cyclic shear stresses applied to the soil by the design earthquake (represented by the cyclic stress ratio, CSR) to the cyclic shear strength offered by the soil (represented as the cyclic resistance ratio, CRR). The CSR is primarily a function of the effective overburden pressure, the design ground acceleration, and the earthquake magnitude specific to the site. The CRR is primarily related to the relative density of the soil and its gradation.

The results of an assessment for this site indicate that the native sands and glacial till soils are in too dense of a state to be potentially liquefiable.

6.4 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the 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 OPSS 1010 Granular A or Granular B Type II (but with less than 5 per cent passing the 200 sieve) should be used as backfill behind the walls. This fill should be placed and compacted in accordance with SP 105S10. 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 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I, Figure C6.20(a) of the Commentary on CHBDC) or within a wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II, Figure C6.20(b) of the Commentary on CHBDC).



- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for the approach embankments:

| Soil Unit Weight | 20 kN/m ³ |
|------------------------------------------------|----------------------|
| Coefficients of static lateral earth pressure: | |
| Active, K_a | 0.35 |
| At rest, K_o | 0.50 |

- For Case II, the pressures are based on the granular fill and the following parameters (unfactored) may be assumed:

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

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows in accordance with Section C6.9.1 of the Commentary to CHBDC:
 - Rotation (i.e., ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem. The stem should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the CHBDC, this site is located in Seismic Performance Zone 3. The site-specific zonal acceleration ratio for Ottawa is 0.2. The seismic lateral earth pressure coefficients given below have therefore been derived based on a design zonal acceleration ratio of $A = 0.2$.



- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e., $k_h = 0.3$). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e., $k_h = 0.1$).
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case II) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

| | Case I | Case II | |
|-------------------|--------|------------|-----------------------|
| | | Granular A | Granular B Type II |
| Yielding wall | 0.39 | 0.30 | 0.30 |
| Non-yielding wall | 0.62 | 0.50 | 0.50 |

- The above K_{AE} values for yielding walls are applicable provided that the calculated wall displacement is more than 250A (mm), where A is the design zonal acceleration ratio of 0.20. This corresponds to displacements of up to approximately 50 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

- Where:
- $\sigma_h(d)$ Is the lateral earth pressure at depth, d, (kPa);
 - K_a Is the static active earth pressure coefficient;
 - K_{AE} Is the seismic active earth pressure coefficient;
 - γ Is the unit weight of the backfill soil (kN/m^3), as given previously;
 - d Is the depth below the top of the wall (m); and,
 - H Is the total height of the wall (m).

6.5 Approach Embankment Design and Construction

Only the east side of the north approach embankment is planned to be widened. The existing east slope has an inclination of approximately 3.7 horizontal to 1 vertical. The additional fill required for this widening is anticipated to be minor, at less than 1 m in height. For the granular foundation soils on this site, only minor/negligible settlements are expected and will largely occur during construction.



6.5.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil and softened/loosened soils be stripped from below the widened approach embankment areas (including the slope area and any new footprint beyond the existing embankment toe), to minimize differential settlement between the existing and widened portions of the approach embankments.

Embankment fill should be placed in regular lifts with a loose thickness not exceeding 300 mm, and be compacted to at least 95 percent of the material's Standard Proctor maximum dry density.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 percent of the Standard Proctor maximum dry density in accordance with MTO Special Provision 105S10. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

To reduce surface water erosion on the widened embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.5.2 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the approach embankment widening with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) or shallower will have a factor of safety of greater than 1.3 against deep-seated slope instability under static loading conditions.

The slope stability analyses for this embankment configuration were carried out using the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.

| Material | Bulk Unit Weight | Effective Angle of Friction |
|--------------------------------------------------------------------------------------------|---------------------------|-----------------------------|
| Native Compact Sands | 20 kN/m ³ | 33° |
| New embankment fill for widening (range of parameters for earth fill and granular fill) | 20 - 22 kN/m ³ | 32° to 35° |
| Compact to very dense sand and gravel (Glacial Till) | 22 kN/m ³ | 43° |
| Limestone Bedrock | - | Impenetrable |

Pseudo-static seismic slope stability analyses for the above configurations also indicate that the embankment side slopes will have factors of safety of greater than 1.1 against deep-seated slope instability based on an acceleration of 0.1g. The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes.



6.6 Construction Considerations

6.6.1 Excavations and Temporary Excavation Support

It is anticipated that excavations for the structure rehabilitation and for replacement wing walls/retaining walls will be advanced through fill materials and the compact native sands. Where space permits, open-cut excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The available information indicates that the fill materials and native sands at the site are classified as Type 3 soils, according to the OHSA. Temporary excavations (i.e., those which are only open for a relatively short period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) through these materials. If excavations below about Elevation 121 m are planned, appropriate dewatering or groundwater control measures will be required; otherwise much flatter side slopes could be needed.

It is expected that temporary roadway protection will be required along the east and west sides of Carp Road, adjacent to the abutments, to facilitate the abutment modifications. These temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of temporary shoring systems should meet Performance Level 2 as specified in SP 105S19, provided that any utilities that may be present adjacent to the temporary shoring systems can tolerate this level of deformation.

Based on the subsurface conditions at the site and the likely excavation geometry, roadway protection (i.e., shoring) could consist of soldier piles and lagging or interlocking steel sheet piling. Cantilevered shoring could likely be used for excavations up to about 3 metres deep. Deeper excavations would require some form of lateral restraint such as internal bracing or rakers, or rock anchor tie-backs.

6.6.2 Groundwater and Surface Water Control

The measured groundwater level at the site varied between Elevations 118.1 to 120.7 m in the open boreholes (from both the 1971 and 2007 soil investigations, with no obvious change over that time). Excavations behind the existing abutments are not therefore anticipated to encounter groundwater. However any excavations at the center piers would likely encounter groundwater. Appropriate groundwater control measures would be required to allow excavation through the water-bearing native till.

Excavations below about Elevation 121 m at the abutment areas, though not anticipated at this time, would extend below the groundwater level in permeable sandy soils. Dewatering of the sand in advance of excavation would be required. Otherwise excavations could encounter excessive groundwater inflow which could disturb the subgrade, destabilize the excavation side slopes, and interfere with the work.

Surface water should be directed away from the excavation area, to prevent ponding of water that could result in disturbance and weakening of the foundation subgrade. Appropriate surface water control should be implemented during construction.



7.0 CLOSURE

This report was prepared by the Project Manager, Mr. Michael Cunningham P.Eng. This report was reviewed by Mr. Fintan J. Heffernan P.Eng, the designated MTO contact for this project.

Yours truly,

GOLDER ASSOCIATES LTD.

Mike Cunningham, P.Eng.
Associate

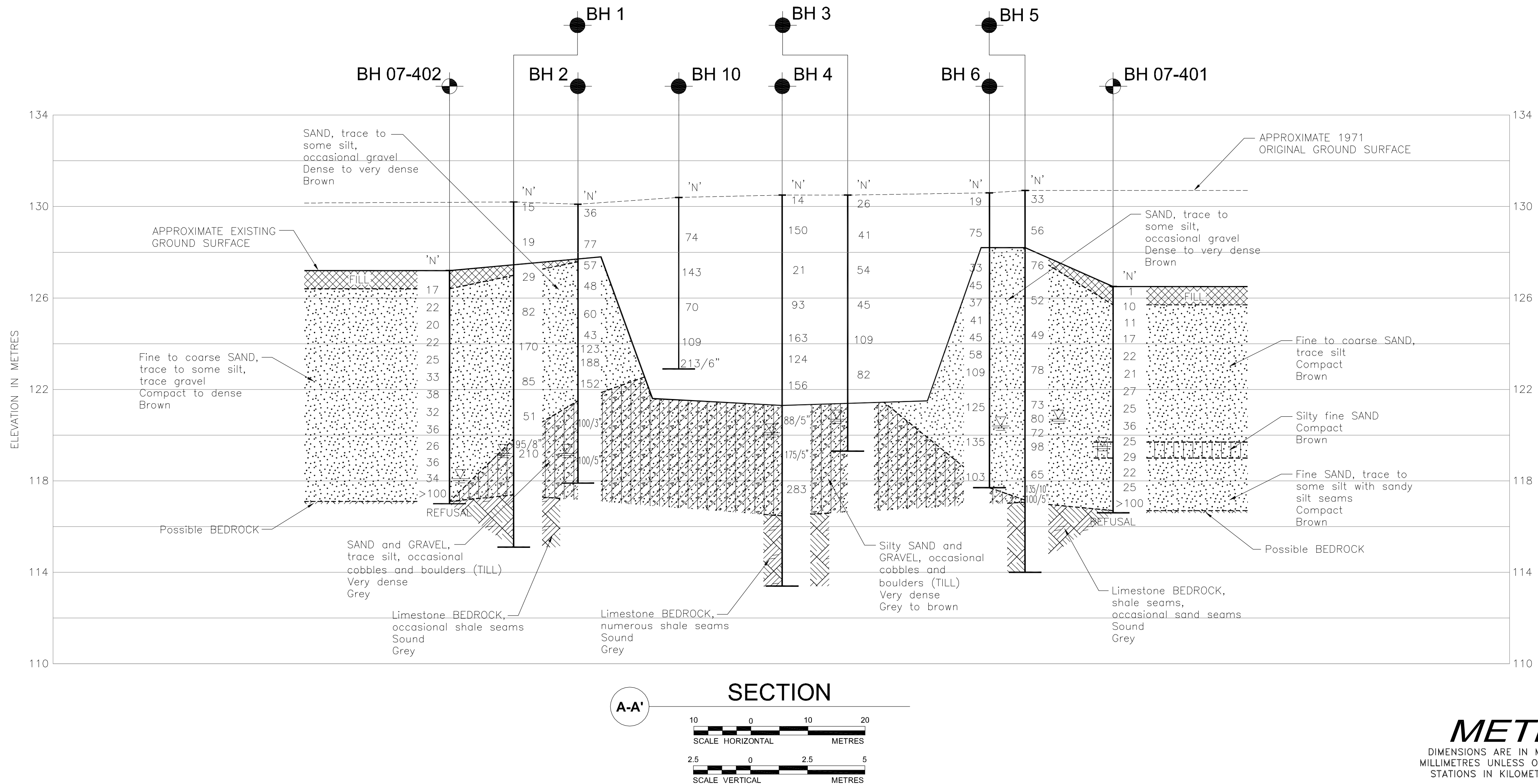
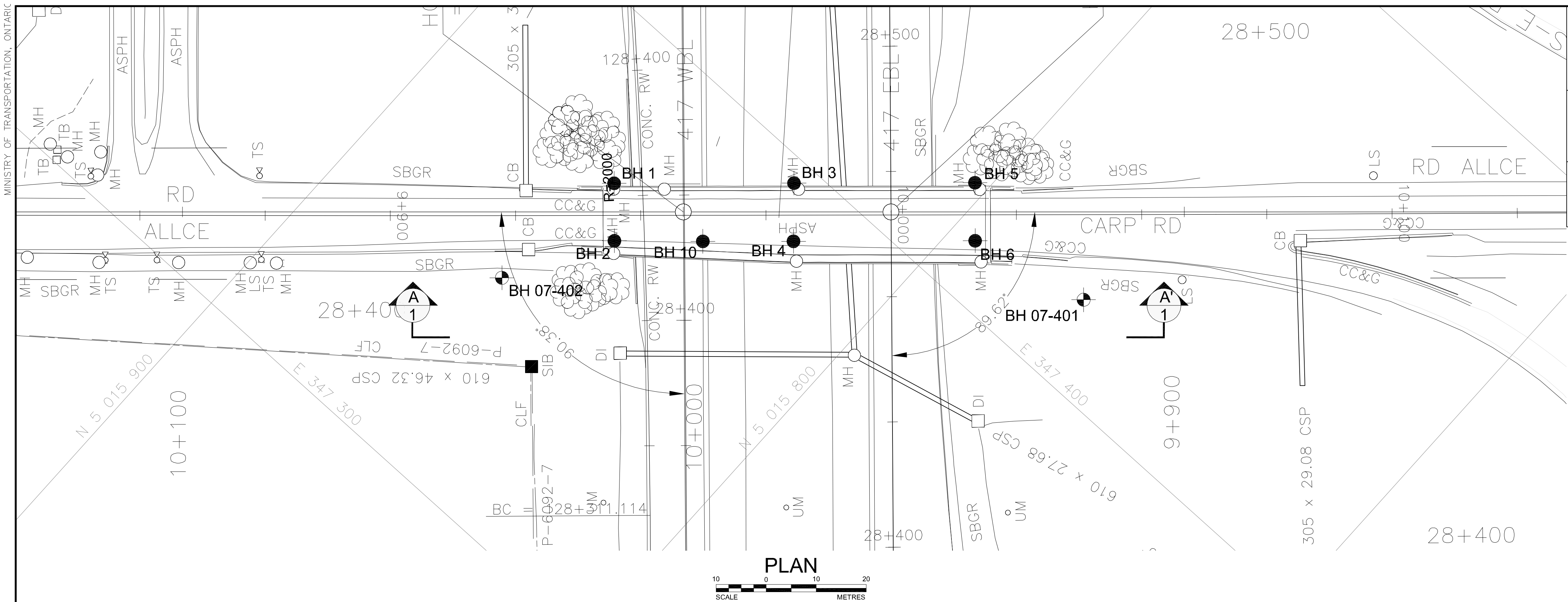


Fintan Heffernan, P.Eng.
Designated MTO Contact



BDG/MIC/FJH/am/cg

n:\active\2007\1121 - geotechnical\07-1121-0151 mrc hwy 417 ottawa\foundation engineering\leagleson & carp road\carp rd underpass\07-1121-0151 carp rd underpass rpt 06july10.docx



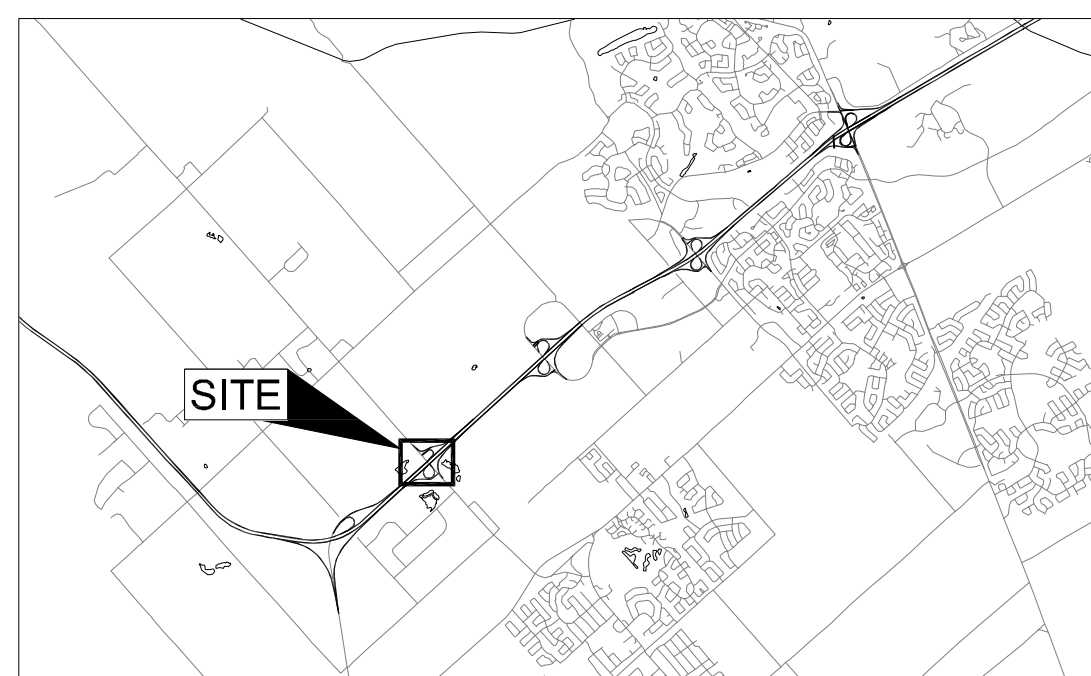
WP No. 255-98-00

**CARP ROAD
BOREHOLE LOCATIONS
AND SOIL STRATA**

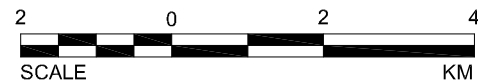
SHEET
1



Golder Associates Ltd.
OTTAWA, ONTARIO, CANADA



KEY PLAN



LEGEND

- Borehole — Current Golder Associates Ltd. Investigation
- Borehole — Previous MTO Investigation Geocres No. 31G5-102
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- Location of cross-section

| No. | ELEVATION | CO-ORDINATES | |
|--------|-----------|--------------|----------|
| | | NORTHING | EASTING |
| 07-401 | 126.5 | 5015771.0 | 347417.0 |
| 07-402 | 127.2 | 5015860.2 | 347342.6 |
| 1 | 130.2 | 5015856.2 | 347371.6 |
| 2 | 130.1 | 5015848.5 | 347363.1 |
| 3 | 130.5 | 5015829.5 | 347395.6 |
| 4 | 130.5 | 5015821.8 | 347386.9 |
| 5 | 130.7 | 5015802.7 | 347419.9 |
| 6 | 130.6 | 5015794.9 | 347411.3 |
| 10 | 130.4 | 5015835.3 | 347374.8 |

REFERENCE

Base plan supplied by the McCormick Rankin Corporation

NOTES

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.

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

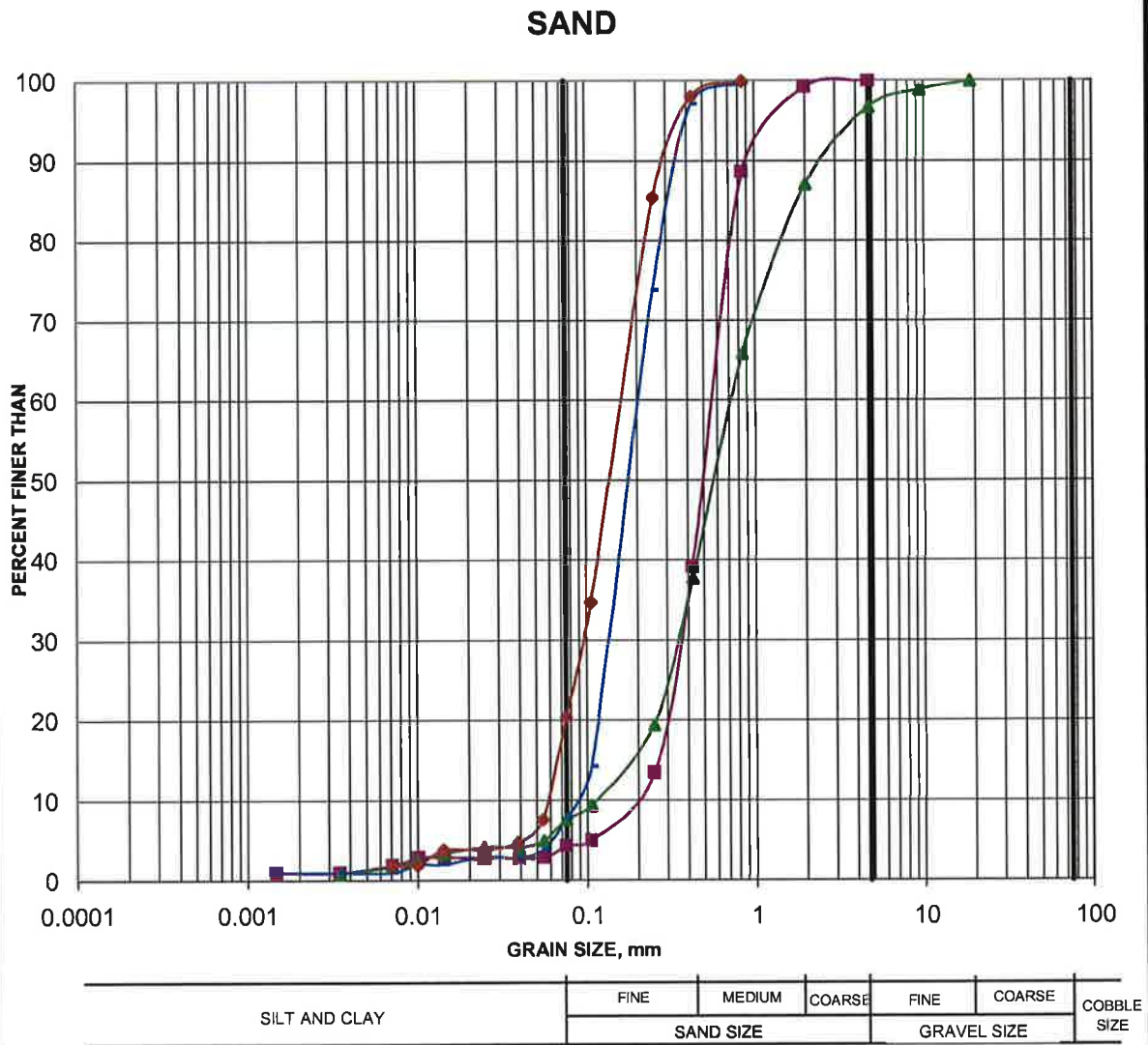
| NO. | DATE | BY | REVISION | |
|----------------------|------|-------------------------|-----------------|-------------|
| Geocres No. 31G5-221 | | | | |
| HWY. 417 | | PROJECT NO.07-1121-0151 | | DIST. 42 |
| SUBM'D. B.D.G. | | CHKD. B.D.G. | DATE: APR. 2008 | SITE: 3-287 |
| DRAWN: J.M. | | CHKD. M.I.C. | APPD. M.I.C. | DWG. 1 |

METRIC

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

GRAIN SIZE DISTRIBUTION

FIGURE 1



| Borehole | Sample | Depth (m) |
|----------|--------|-----------|
| 07-401 | 3 | 1.22-1.83 |
| 07-401 | 5 | 2.74-3.35 |
| 07-401 | 7 | 4.27-4.88 |
| 07-401 | 9 | 5.79-6.40 |



APPENDIX A

List of Abbreviations and Symbols Records of Borehole Sheets 07-401 and 07-402

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 | | III. SOIL DESCRIPTION | |
|----------------------------------------------------------|-----------------------------------------------|-----------------------|-----------------------------------------------------------------------------------------------------|
| AS | Auger sample | (a) | Cohesionless Soils |
| BS | Block sample | | |
| CS | Chunk sample | Density Index | N |
| DO | Drive open | (Relative Density) | Blows/300 mm |
| DS | Denison type sample | | Or Blows/ft. |
| FS | Foil sample | Very loose | 0 to 4 |
| RC | Rock core | Loose | 4 to 10 |
| SC | Soil core | Compact | 10 to 30 |
| ST | Slotted tube | Dense | 30 to 50 |
| TO | Thin-walled, open | Very dense | over 50 |
| TP | Thin-walled, piston | | |
| WS | Wash sample | (b) | Cohesive Soils |
| II. PENETRATION RESISTANCE | | Consistency | $C_{u2}S_u$ |
| Standard Penetration Resistance (SPT), N: | | | Kpa |
| The number of blows by a 63.5 kg. (140 lb.) | | Very soft | 0 to 12 |
| hammer dropped 760 mm (30 in.) required | | Soft | 12 to 25 |
| to drive a 50 mm (2 in.) drive open | | Firm | 25 to 50 |
| Sampler for a distance of 300 mm (12 in.) | | Stiff | 50 to 100 |
| DD- Diamond Drilling | | Very stiff | 100 to 200 |
| Dynamic Penetration Resistance; N_d: | | Hard | Over 200 |
| The number of blows by a 63.5 kg (140 lb.) | | | Psf |
| hammer dropped 760 mm (30 in.) to drive | | | 0 to 250 |
| Uncased a 50 mm (2 in.) diameter, 60° cone | | | 250 to 500 |
| attached to "A" size drill rods for a distance | | | 500 to 1,000 |
| of 300 mm (12 in.). | | | 1,000 to 2,000 |
| | | | 2,000 to 4,000 |
| | | | Over 4,000 |
| PH: | Sampler advanced by hydraulic pressure | IV. SOIL TESTS | |
| PM: | Sampler advanced by manual pressure | w | water content |
| WH: | Sampler advanced by static weight of hammer | w_p | plastic limited |
| WR: | Sampler advanced by weight of sampler and rod | 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 test (LV-laboratory vane test) |
| | | γ | unit weight |

Note:

- Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

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

I. GENERAL

| | |
|---------------------------|-----------------------------|
| π | = 3.1416 |
| $\ln x$ | natural logarithm of x |
| $\log_{10} x$ or $\log x$ | logarithm of x to base 10 |
| g | Acceleration due to gravity |
| t | time |
| F | factor of safety |
| V | volume |
| W | weight |

II. STRESS AND STRAIN

| | |
|------------------------------|----------------------------------------------------------------------------|
| γ | shear strain |
| Δ | change in, e.g. in stress: $\Delta \sigma'$ |
| ϵ | linear strain |
| ϵ_v | volumetric strain |
| η | coefficient of viscosity |
| ν | Poisson's ratio |
| σ | total stress |
| σ' | effective stress ($\sigma' = \sigma - u$) |
| σ'_{vo} | initial effective overburden stress |
| $\sigma_1 \sigma_2 \sigma_3$ | principal stresses (major, intermediate, minor) |
| σ_{oct} | mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$ |
| τ | shear stress |
| u | porewater pressure |
| E | modulus of deformation |
| G | shear modulus of deformation |
| K | bulk modulus of compressibility |

III. SOIL PROPERTIES

(a) Index Properties

| | |
|--------------------|-------------------------------------------------------------------------------------------------------------------------------------|
| $\rho(\gamma)$ | bulk density (bulk unit weight*) |
| $\rho_d(\gamma_d)$ | dry density (dry unit weight) |
| $\rho_w(\gamma_w)$ | density (unit weight) of water |
| $\rho_s(\gamma_s)$ | density (unit weight) of solid particles |
| γ' | unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$) |
| D_R | relative density (specific gravity) of solid particles ($D_R = \rho_s/\rho_w$) formerly (G_s) |
| e | void ratio |
| n | porosity |
| S | degree of saturation |
| * | Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity) |

(a) Index Properties (cont'd.)

| | |
|-----------|------------------------------------------------------------------------------------|
| 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 (overconsolidated range) |
| C_s | swelling index |
| C_a | coefficient of secondary consolidation |
| m_v | coefficient of volume change |
| c_v | coefficient of consolidation |
| T_v | time factor (vertical direction) |
| U | degree of consolidation |
| σ'_p | pre-consolidation pressure |
| OCR | Overconsolidation ratio = σ'_p/σ'_{vo} |

(d) Shear Strength

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

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

2. Shear strength = (Compressive strength)/2

| PROJECT 07-1121-0151 | | | RECORD OF BOREHOLE No 07-401 | | | 1 OF 1 | | | METRIC | | | | | | | | | | | | | | | | | |
|----------------------|---------------------------------------------------------------------------------------|------------|--------------------------------------------------|------|------------|-------------------------|--|--|-----------------|--|--|------------------------------------------|--|--|-----------------------------------------------------|--|--|-------------------|--|--|-------------|--|--|---------------------------------------|--|--|
| W.P. 255-98-00 | | | LOCATION N 5015771.0; E 347417.0 | | | ORIGINATED BY D.J.S. | | | | | | | | | | | | | | | | | | | | |
| DIST HWY 417 | | | BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem | | | COMPILED BY J.M. | | | | | | | | | | | | | | | | | | | | |
| DATUM Geodetic | | | DATE Nov. 14, 2007 | | | CHECKED BY S.A.T. | | | | | | | | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | | | ELEVATION SCALE | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | WATER CONTENT (%) | | | UNIT WEIGHT | | | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | | | | | | | | | | | | | | | | | | | |
| 126.5 | GROUND SURFACE | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.0 | Sandy topsoil (FILL) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.2 | Dark brown Moist | | 1 | SS | 1 | | | | | | | | | | | | | | | | | | | | | |
| 125.7 | Fine to coarse sand (FILL) | | | | | | | | | | | | | | | | | | | | | | | | | |
| 0.8 | Very loose Brown Moist | | 2 | SS | 10 | | | | | | | | | | | | | | | | | | | | | |
| | Fine to medium SAND | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Compact Brown Moist | | 3 | SS | 11 | | | | | | | | | | | | | | | | | | | | | |
| 124.2 | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 2.3 | Fine to coarse SAND, trace gravel | | 4 | SS | 17 | | | | | | | | | | | | | | | | | | | | | |
| | Compact Brown Moist | | | | | | | | | | | | | | | | | | | | | | | | | |
| 123.3 | | | 5 | SS | 22 | | | | | | | | | | | | | | | | | | | | | |
| 3.2 | Stratified fine SAND, trace silt | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Compact to dense Light brown Moist | | 6 | SS | 21 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | 7 | SS | 27 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | 8 | SS | 25 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | 9 | SS | 36 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| 119.7 | | | 10 | SS | 25 | | | | | | | | | | | | | | | | | | | | | |
| 6.8 | Silty fine SAND | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Compact Brown Moist to wet | | | | | | | | | | | | | | | | | | | | | | | | | |
| 119.0 | | | 11 | SS | 29 | | | | | | | | | | | | | | | | | | | | | |
| 7.5 | Fine SAND, trace to some silt with sandy silt seams | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Compact Brown Wet | | 12 | SS | 22 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | 13 | SS | 25 | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | | | 14 | SS | >100 | | | | | | | | | | | | | | | | | | | | | |
| 116.7 | Possibly weathered BEDROCK | | | | | | | | | | | | | | | | | | | | | | | | | |
| 9.9 | End of Borehole Auger Refusal | | | | | | | | | | | | | | | | | | | | | | | | | |
| | Note: Groundwater encountered at approx. 8.4m depth during drilling on Nov. 14, 2007. | | | | | | | | | | | | | | | | | | | | | | | | | |

MISS MTO 07-1121-0151 GPJ ON MOT GDT 9/5/08

| PROJECT 07-1121-0151 | | | RECORD OF BOREHOLE No 07-402 | | | 1 OF 1 | | | METRIC | | | | |
|----------------------|----------------------------------|------------|--------------------------------------------------|------|------------|------------------------------------------|-----------------|-----------------|---------------|--------------------------|--------------|-------------|---------------------------------------|
| W.P. 255-98-00 | | | LOCATION N 5015860.2; E 347342.6 | | | ORIGINATED BY D.J.S. | | | | | | | |
| DIST HWY 417 | | | BOREHOLE TYPE Power Auger 108mm I.D. Hollow Stem | | | COMPILED BY J.M. | | | | | | | |
| DATUM Geodetic | | | DATE Nov. 13, 2007 | | | CHECKED BY S.A.T. | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | GROUND WATER CONDITIONS | ELEVATION SCALE | 20 40 60 80 100 | PLASTIC LIMIT | NATURAL MOISTURE CONTENT | LIQUID LIMIT | UNIT WEIGHT | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
| 127.2 | GROUND SURFACE | | | | | | | | | | | | |
| 0.0 | Sandy topsoil (FILL) | | | | | | 127 | | | | | | |
| 0.2 | Dark brown | | | | | | | | | | | | |
| 126.4 | Fine sand (FILL) | | | | | | | | | | | | |
| 0.8 | Brown | | | | | | | | | | | | |
| 125.2 | Stratified fine SAND, trace silt | | 1 | SS | 17 | | 126 | | | | | | |
| 2.0 | Compact | | 2 | SS | 22 | | | | | | | | |
| 123.5 | Light brown | | 3 | SS | 20 | | 125 | | | | | | |
| 123.5 | Moist | | 4 | SS | 22 | | 124 | | | | | | |
| 122.3 | Stratified fine SAND | | 5 | SS | 25 | | 123 | | | | | | |
| 120.4 | Compact | | 6 | SS | 33 | | 122 | | | | | | |
| 119.7 | Brown | | 7 | SS | 38 | | 121 | | | | | | |
| 118.7 | Moist | | 8 | SS | 32 | | 120 | | | | | | |
| 117.1 | Stratified fine SAND, trace silt | | 9 | SS | 36 | | 119 | | | | | | |
| 117.1 | Dense | | 10 | SS | 26 | | 118 | | | | | | |
| 117.1 | Light brown | | 11 | SS | 36 | | | | | | | | |
| 117.1 | Moist | | 12 | SS | 34 | | | | | | | | |
| 117.1 | Fine SAND, trace to some silt | | 13 | SS | >100 | | 117 | | | | | | |
| 117.1 | Dense | | | | | | | | | | | | |
| 117.1 | Brown to grey | | | | | | | | | | | | |
| 117.1 | Wei | | | | | | | | | | | | |
| 117.1 | Possibly weathered BEDROCK | | | | | | | | | | | | |
| 117.1 | End of Borehole | | | | | | | | | | | | |
| 117.1 | Auger Refusal | | | | | | | | | | | | |
| 117.1 | Note: | | | | | | | | | | | | |
| 117.1 | Groundwater encountered | | | | | | | | | | | | |
| 117.1 | at approx. 7.7m depth during | | | | | | | | | | | | |
| 117.1 | drilling on Nov. 13, 2007. | | | | | | | | | | | | |

MISS_MTO 07-1121-0151.GPJ ON_MOT GDT 9/5/08



APPENDIX B

Record of Borehole Sheets, 1971 Investigation by MTO
Grain Size Distributions from 1971 Investigation by MTO

Dist. 9

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & TESTING OFFICE
 JOB 71-11019 LOCATION Sta. 99 + 55, o/s 19' lt. (Reg. Rd. #5)
 W.P. 437-64-00 BORING DATE April 19, 20 and 21, 1971
 DATUM Geodetic BOREHOLE TYPE Mashborinq - NX, BX Casing - BX Rock Corechecked BY

RECORD OF BOREHOLE No. 1 FOUNDATION SECTION

Dynamic Cone Penetration Test

| ELEV. DEPTH | SOIL PROFILE DESCRIPTION | STRAT. PLOT | SAMPLES | | ELEV. SCALE | DYNAMIC PENETRATION BLOWS / FOOT | | SHEAR STRENGTH - PSF | | PLASTIC LIMIT WATER CONTENT % | | REMARKS |
|----------------|-----------------------------------------------------------------------------------------|-------------|---------|----------|-------------|-------------------------------------|----|----------------------|----|----------------------------------|----|-----------------------------|
| | | | NUMBER | TYPE | | 60 | 80 | 100 | 10 | 20 | 30 | |
| 427.1 | Ground Level | | 1 | SS 15 | | | | | | | | |
| 425.0 | Org. matter (fill), compact | | 2 | SS 19 | | | | | | | | |
| 420.0 | Boulders up to 6" in size | | 3 | SS 29 | | | | | | | | |
| 410.0 | sand, trace to some silt, occasional gravel (uniformly graded - irregularly stratified) | | 4 | SS 82 | | | | | | | | 0 96 (4) |
| 400.0 | Brown | | 5 | SS 170 | | | | | | | | |
| 390.0 | Compact to v. dense | | 6 | SS 85 | | | | | | | | 6 80 (14) |
| 385.0 | Met. Mix. sand & gra. trace of silt | | 7 | SS 51 | | | | | | | | 391.0 |
| 380.0 | Glacial Till (ocg) in boulders up to 6" in size (throughout) | | 8 | SS 9572" | | | | | | | | 54 34 (12) |
| 375.0 | Limestone Bedrock, occ. shaly seams (Random ss. seams up to 1" thick) | | 9 | SS 210 | | | | | | | | W.L. in open BH, Apr. 30/71 |
| 370.0 | Grey sand | | 10 | BX 492 | | | | | | | | |
| 365.0 | | | 11 | BX 153 | | | | | | | | |
| 360.0 | | | 12 | PX 254 | | | | | | | | |
| 355.0 | | | 13 | BX 802 | | | | | | | | |
| 350.0 | | | 14 | BX 753 | | | | | | | | |
| 345.0 | End of Borehole | | | | | | | | | | | |

Practical Refusal
 Elev. 407.2

| DEPARTMENT OF HIGHWAYS- ONTARIO MATERIALS & TESTING OFFICE | | RECORD OF BOREHOLE No. 2 | | | | FOUNDATION SECTION | |
|---------------------------------------------------------------|-------------|--------------------------------------------------|---------------------------------|------------------------------------------------|------------------------|-----------------------------|-------------------|
| JOB W.O. 71-11019 | | LOCATION Sta. 99 + 55, o/s 19' rt. (Reg. Rd. #5) | | ORIGINATED BY B.T.D. | | | |
| W.P. 437-61-00 | | BORING DATE April 26, 28, and 29, 1971 | | COMPILED BY A.R.D. | | | |
| DATUM Geodetic | | BOREHOLE TYPE Washboring - WX, BX Casing | | CHECKED BY | | | |
| SOIL PROFILE | | DYNAMIC Cone Penetration Test | | LIQUID LIMIT PLASTIC LIMIT WATER CONTENT | | REMARKS | |
| ELEV. DEPTH | DESCRIPTION | SAMPLES NUMBER | DYNAMIC PENETRATION BLOWS/FC | RESISTANCE PSF | FIELD VANE LAB VANE | WATER CONTENT % 10 20 30 | PC.F GR.SA.SI.CL. |
| 427.0 Ground Level | | | | | | | |
| 425.0 (fill), compact | | 1 SS 36 | 20 | 60 | | | |
| 2.0 Sand, trace to some silt, occasional gravel throughout | | 2 SS 77 | 420 | | | | |
| (Uniformly graded + irregularly stratified) | | 3 SS 57 | | | | | |
| | | 4 SS 48 | | | | | |
| | | 5 SS 60 | 410 | | | | |
| | | 6 SS 43 | | | | | |
| | | 7 SS 123 | | | | | |
| | | 8 SS 178 | | | | | |
| | | 9 SS 152 | 400 | | | | |
| 400.0 Dense to v. dense | | 10 BX 144 | | | | | |
| 27.0 Het. Mix. of sand & gravel, trace of sil., Glacial Till | | 11 SS 100/3 | | | | | |
| (occ. boulders up to 7 in. in size) | | 12 BX 91 | | | | | |
| | | 13 SS 100/5 | | | | | |
| 397.0 Grey v. dense | | 14 BX 124 | | | | | |
| 40.0 End of Borehole | | | | | | | |

Practical Refusal
Elev. 407.0

31 40 (20)

391.0
open
Apr. 30/71

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

FOUNDATION SECTION

JOB 71-11019

LOCATION Sta. 100 + 73, o/s 19' Lt. (Reg. Rd. #5)

ORIGINATED BY B.T.D.

W.P. 437-64-00

BORING DATE April 21, 22, and 23, 1971

COMPILED BY A.E.D.

DATUM Geodetic

BOREHOLE TYPE Washboring - NX, BX, AX Casings

CHECKED BY

| SOIL PROFILE | | SAMPLES | | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT | | LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w | | BULK DENSITY γ | REMARKS |
|-------------------------|--------------------------------------------------------------------------------------|---------|-------------|-------------|------------------------------------------------|------------------------------|----------------------------------------------------------------------|-----------------------------|--------------------------|---------------------------------------------------------------------------------|
| ELEV. DEPTH | DESCRIPTION | NUMBER | TYPE | | SHEAR STRENGTH P.S.F. | UNCONFINED QUICK TRIAXIAL | FIELD VANE LAB. VANE | WATER CONTENT % 10 20 30 | | |
| 428.0 | Ground Level | 1 | SS 26 | | | | | | | GR. SA. SI. CL. 14 36 (50) 396.0 W.L. in open BH Apr. 30/71 |
| 0.05a.; some gra. & sl. | compact | 2 | SS 41 | | | | | | | |
| 425.0 (Fill) | | 3 | SS 54 | 420 | | | | | | |
| 3.0 | Sand, trace to some silt, occasional gravel | 4 | SS 45 | | | | | | | |
| | (Uniformly graded - irregularly stratified) | 5 | SS 109 | 410 | | | | | | |
| | Brown | 6 | SS 82 | | | | | | | |
| 402.5 | Dense to v. dense | 7 | BX 85% Rec | 400 | | | | | | |
| 25.5 | Het. Mix. of silt, sa. & gra., Glacial Till, (Boulders up to 16" in size throughout) | 8 | AXT 47% Red | | | | | | | |
| 391.3 | (Grey) v. dense | | | 390 | | | | | | |
| 36.7 | End of Borehole | | | | | | | | | |

14 36 (50)

396.0
W.L. in open BH
Apr. 30/71

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

FOUNDATION SECTION

JOB 71-11019

LOCATION Sta. 100 + 73, o/s 19' rt. (Reg. Rd. #5)

ORIGINATED BY J.D.W.

W.P. 437-64-00

BORING DATE April 21, 22, 26 and 27, 1971

COMPILED BY B.T.D.

DATUM Geodetic

BOREHOLE TYPE Washboring - NX, BX, AX Casing

CHECKED BY

BX and AXT Rock Core

| SOIL PROFILE | | SAMPLES | | | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE | | LIQUID LIMIT — w_L | | | BULK DENSITY | REMARKS |
|--------------|------------------------------------------------------------------------------------|-------------|--------|------|-------------|--------------------------------|------------|-----------------------|---------------------|-----------------|--------------|---------|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | | BLOWS / FOOT | RESISTANCE | PLASTIC LIMIT — w_p | WATER CONTENT — w | WATER CONTENT % | | |
| 428.1 | Ground Level | | | | | | | | | | | |
| 425.1 | C. 0.5a some prs. & sl. (Fill) Brown, Compact | | 1 | SS | 14 | | | | | | | |
| 3.0 | Boulders up to 9" in size | | 2 | SS | 150/5" | | | | | | | |
| | | | 3 | BX | 38% | | | | | | | |
| | sand, trace to some silt, occasional gravel sizes (Uniform-irregularly stratified) | | 4 | SS | 21 | | | | | | | |
| | Brown | | 5 | SS | 93 | | | | | | | |
| 404.4 | Compact to v. dense | | 6 | SS | 163 | | | | | | | |
| 23.7 | Het. Mix. of silt, sand & gravel | | 7 | SS | 124 | | | | | | | |
| | Glacial Till | | 8 | BX | 100% | | | | | | | |
| | Very bouldery throughout-boulders up to 10" in size | | 9 | BX | 65% | | | | | | | |
| | Grey to Brown | | 10 | SS | 156 | | | | | | | |
| | very dense | | 11 | BX | 39% | | | | | | | |
| | | | 12 | SS | 66/5" | | | | | | | |
| | | | 13 | BX | 50% | | | | | | | |
| | | | 14 | SS | 175/5" | | | | | | | |
| | | | 15 | BX | 23% | | | | | | | |
| | | | 16 | SS | 283 | | | | | | | |
| 382.1 | | | 17 | AXT | 56% Rec | | | | | | | |
| 46.0 | Limestone Bedrock numerous shale seams | | 18 | AXT | 100% Red | | | | | | | |
| | Grey | | | | 95% | | | | | | | |
| 372.1 | Sound | | 19 | AXT | Rec | | | | | | | |
| 56.0 | End of Borehole | | | | | | | | | | | |

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

FOUNDATION SECTION

JOB 71-11019

LOCATION Sta. 101 + 91, o/s 19' lt. (Reg Rd. #5)

ORIGINATED BY B.T.D.

W.P. 437-64-00

BORING DATE April 19, 20, and 21, 1971

COMPILED BY A.E.D.

DATUM Geodetic

BOREHOLE TYPE NX, BX Casing - BX Rock Core
Dynamic Cone Penetration Test

CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | ELEV. SCALE | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT | | LIQUID LIMIT ——— w _L PLASTIC LIMIT ——— w _p WATER CONTENT ——— w | | BULK DENSITY γ | REMARKS | |
|--------------|-------------------------------------------------------------------|-------------|---------|------|--------------|-------------|------------------------------------------------|----|--------------------------------------------------------------------------------------------|----|-------------------|---------|-----------|
| ELEV. DEPTH | DESCRIPTION | STRAT. PLOT | NUMBER | TYPE | BLOWS / FOOT | | 20 | 40 | 60 | 80 | | | 100 |
| 428.8 | Ground Level | | | | | | | | | | | | |
| 427.3 | ss, some gra. (fill) | | 1 | SS | 33 | | | | | | | | |
| 1.5 | Sand, trace to to some silt occasional gravel throughout) | | 2 | SS | 56 | 420 | | | | | | | |
| | | | 3 | SS | 76 | | | | | | | | |
| | Brown | | 4 | SS | 52 | 410 | | | | | | | 0 86 (14) |
| | Dense to very dense | | 5 | SS | 49 | | | | | | | | |
| | | | 6 | SS | 78 | 400 | | | | | | | 0 88 (12) |
| | | | 7 | SS | 73 | | | | | | | | |
| | | | 8 | SS | 80 | | | | | | | | |
| | | | 9 | SS | 78 | | | | | | | | |
| | | | 10 | SS | 98 | 390 | | | | | | | |
| | | | 11 | SS | 65 | | | | | | | | |
| 385.3 | sal. brown, v. dense | | 12 | SS | 135 | 380 | | | | | | | |
| 384.7 | glacial till, v. | | 13 | SS | 135 | | | | | | | | |
| 44.7 | Limestone Bedrock, seams of shale, occ. sand seams up to 2" thick | | 14 | BX | 69% | | | | | | | | |
| | | | 15 | BX | 75% | 380 | | | | | | | |
| | | | 16 | BX | 85% | | | | | | | | |
| 374.1 | Grey Sand | | 17 | BX | 71% | | | | | | | | |
| 54.7 | End of Borehole | | | | | 370 | | | | | | | |

| | | | | | | | | |
|-----------------------------------------|--|--|---------------------------------------------------|--|--|--------------------------------|--|--|
| DEPARTMENT OF HIGHWAYS- ONTARIO | | | RECORD OF BOREHOLE No. 6 | | | FOUNDATION SECTION | | |
| MATERIALS & TESTING OFFICE | | | LOCATION Sta. 107 + 91, o/s 19' rt. (Rep. Rd. #5) | | | ORIGINATED BY J.D.W. | | |
| JOB 71-11019 | | | BORING DATE April 23, and 26, 1971 | | | COMPILED BY A.E.D. | | |
| WP 437-64-00 | | | BOREHOLE TYPE NX Casings | | | CHECKED BY | | |
| DATUM Geodetic | | | Dynamic Cone Penetration Test | | | | | |
| SOIL PROFILE | | | SAMPLES | | | DYNAMIC PENETRATION RESISTANCE | | |
| ELEV. DEPTH | | | NUMBER TYPE | | | BLOWS/FOOT | | |
| 420.5 Ground Level | | | 1 SS 19 | | | 20 40 60 80 100 | | |
| 420.5 (11' High) of grade | | | 2 SS 75 | | | SHEAR STRENGTH PSF | | |
| 2.0 Sand, trace to some silt, ccc. | | | | | | O UNCONFINED + FIELD VANE | | |
| gravel | | | | | | ● QUICK TRIAXIAL x LAB. VANE | | |
| uniformly graded-irregularly stratified | | | 3 SS 32 | | | WATER CONTENT % | | |
| Brown | | | 4 SS 45 | | | w _L w _p | | |
| Dense to v. dense | | | 5 SS 37 | | | PLASTIC LIMIT | | |
| | | | 6 SS 41 | | | WATER CONTENT | | |
| | | | 7 SS 45 | | | WATER CONTENT | | |
| | | | P SS 58 | | | WATER CONTENT | | |
| | | | 9 SS 109 | | | WATER CONTENT | | |
| | | | 10 SS 125 | | | WATER CONTENT | | |
| | | | 11 SS 135 | | | WATER CONTENT | | |
| | | | 12 SS 103 | | | WATER CONTENT | | |
| 326.3 | | | | | | P.C.F. GR.SA.SI.CL. | | |
| 42.2 End of Borehole Probable Bedrock | | | | | | REMARKS | | |
| | | | | | | W.L. 1 in open 30 Apr. 30/71 | | |

420.5

420

410

400

390

380

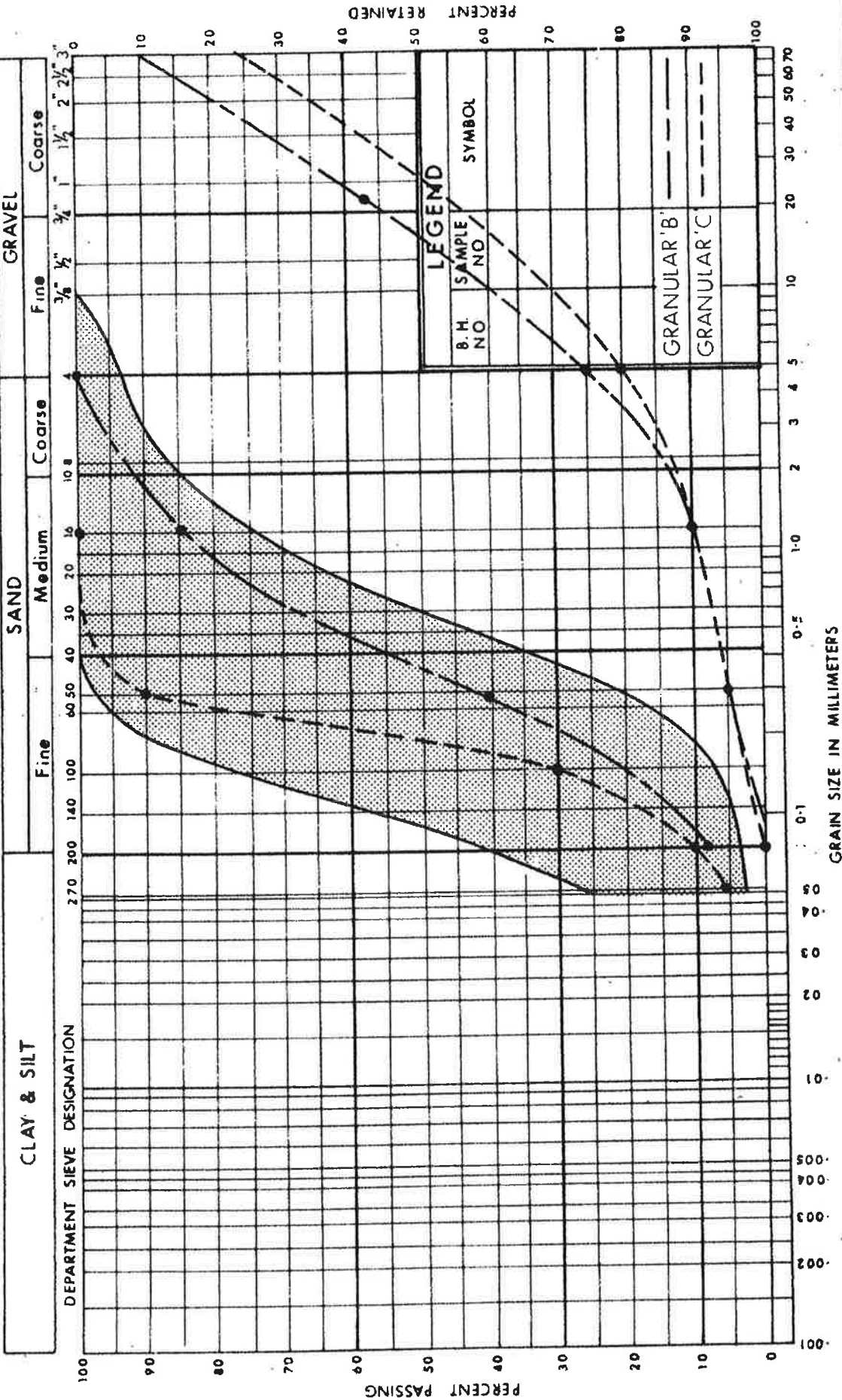
Practical Refusal Elev. 402.5

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

DATUM Geodetic

| ELEV DEPTH | SCH PROFILE DESCRIPTION | STRAT PLOT | SAMPLES NUMBER TYPE BLOWS / FOOT | DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE | LIQUID LIMIT PLASTIC LIMIT WATER CONTENT % w _p w _L w _P | BULK DENSITY γ _c | REMARKS |
|---------------|-------------------------------------------|------------|-------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------|-----------------------------|------------------------------|
| | | | | | w _p → w _L ← w _P | P.C.E. | G.R.S.A. S.I.C.L. |
| 427.6 | Ground Level | [Symbol] | | | | | |
| 425.8 | Top of [illegible] (fill) | [Symbol] | | | | | |
| 2.0 | sand, trace to some silt, occ. gravel | [Pattern] | 1 SS 74 | 420 | | | BH dry, April 29, 1971 |
| | Uniformly graded - irregularly stratified | [Pattern] | 2 SS 74.3 | | | | |
| | brown | [Pattern] | 3 SS 70 | 410 | | | |
| 403.8 | Very dense | [Pattern] | 4 SS 109 | | | | |
| 403.8 | | [Pattern] | 5 SS 213/6" | | | | |
| 24.5 | End of Borehole | [Symbol] | | 400 | | | 42.42 (16) |

UNIFIED SOIL CLASSIFICATION SYSTEM



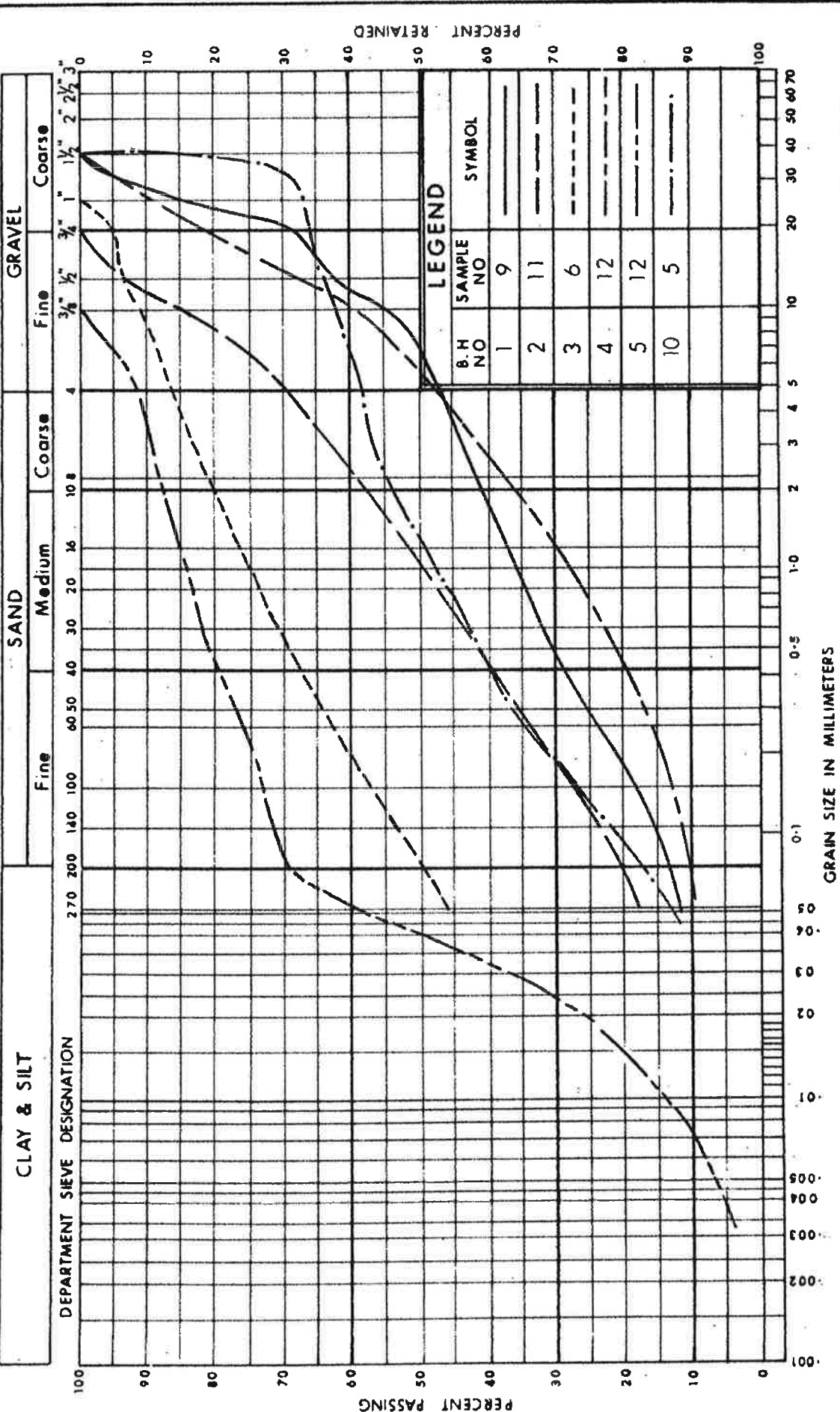
GRAIN SIZE DISTRIBUTION
SAND
TRACE TO SOME GRAVEL

W.P. No. 437-64-00
JOB No: 70-11019
FIG. 1

DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION



UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
HET. MIXTURE OF SILT, SAND & GRAVEL

W.P. No. 437-64-00

JOB No. 70-11019

FIG. 2