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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF CPR OVERHEAD STRUCTURE
ALONG HIGHWAY 6 SOUTH OF HIGHWAY 401
SITE NO. 35-366, G.W.P. 416-98-00

Submitted to:

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GEOCRES NO. 40P8-131

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SITE NO. 35-366, G.W.P. 416-98-00
MINISTRY OF TRANSPORTATION, ONTARIO

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (Morrison Hershfield) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a detail foundation investigation as part of the preliminary design for the replacement of the Canadian Pacific Railway (CPR) overhead structure located on Highway 6 to the south of Highway 401 in Puslinch, Ontario (G.W.P 416-98-00)

The terms of reference for the scope of work are outlined in Golder's proposal P31-1063, dated February 2003, that forms part of the Consultant's Agreement (Number P.O. 3005-A-000281) for this project. The foundation investigation was carried out to meet the requirements for Stage IIB of the Terms of Reference (TOR). The work was carried out in accordance with the Quality Control Plan for this project dated May, 2003. The preliminary general arrangement drawing for the proposed replacement of the bridge structure at the CPR overhead was provided to Golder by Morrison Hershfield in September 2003.

The purpose of the investigation was to establish the subsurface conditions at the area of the proposed bridge abutments, retaining walls, and approach embankments. The specific location of the investigation site is shown in plan on Drawing 1. The investigation was supplemented with information contained in the following report:

- Foundation Investigation Report, Widening of Existing (or Completely New) Overhead, Hwy. No. 6 and C.P.R. at the Village of Puslinch, W.J. 62-F-107, W.P. 75-61, Geocres No. 40P8-29, dated October 1962.

2.0 SITE DESCRIPTION

The existing CPR overhead structure is located on Highway 6, approximately 2.5 km south of Highway 401 in Puslinch Township, Ontario. The existing bridge was constructed in 1928 and the bridge was widened to the west and rehabilitated in 1967. The existing bridge was constructed in fill, about 8 m high, above the existing railway tracks. Entry to the investigation site, immediately west of the existing bridge, was gained via Station Rd. and Fielding Lane, located just north and south of the bridge, respectively.

The terrain in the vicinity of the site generally consists of open fields and pastures, bush areas, shallow rolling hills, and localized swamps. The original ground surface below the existing bridge and embankment slopes ranges from about Elevation 300 m at the centre of the structure to Elevation 301 m at the north and south limits of the site. The CPR tracks are at constructed grade at the bridge. The topography of the region becomes more variable beyond the limits of the site.

3.0 INVESTIGATION PROCEDURES

3.1 Foundation Investigation

The site investigation was carried out from September 29 to October 1, 2003. A total of six (6) boreholes (designated BH03-1 through BH03-6) were advanced on the west side of the existing bridge, both north and south of the CPR tracks as shown on Drawing 1.

The boreholes were drilled as close as possible to the proposed bridge footings; however, due to the presence of numerous underground and overhead utilities, the boreholes could not be drilled in the exact locations of the proposed foundation units. Two (2) boreholes were drilled for the north and south abutment footings. Two (2) boreholes were drilled for the proposed RSS retaining walls located back from the proposed abutments. These boreholes were advanced to auger refusal and were extended by coring a minimum of 3.0 m into the bedrock. Two (2) boreholes were advanced in the areas of the proposed north and south approach embankments. These boreholes were advanced to auger refusal where bedrock was inferred.

All boreholes were drilled using a track-mounted CME 55 drill rig equipped with 108 mm inside diameter (I.D.) hollow stem augers; bedrock was cored using ‘NQ’ coring equipment. The drill rig was supplied and operated by Geo-Environmental Drilling Ltd. Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m in depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures.

The boreholes were advanced to depths ranging from 6.1 m to 10.6 m below the existing ground surface (including rock coring where applicable). The groundwater conditions in the open boreholes were observed during the drilling operations and piezometers were installed in selected boreholes to permit monitoring of the groundwater level at these locations. The piezometers consist of a 25 mm outside diameter rigid PVC tubing with a 0.3 m long slotted tip backfilled with a sand filter and sealed with bentonite within the boreholes. The holes were backfilled with bentonite mixed with soil cuttings; typically one bag of bentonite was used per 3 m of hole backfilled. The installation details and water level readings are described on the Record of Borehole sheets that follow the text of this report.

The field work was supervised by members of our engineering and technical staff, who located the boreholes, arranged for the clearance of underground service locations, supervised the drilling, sampling and in-situ testing operations, logged the boreholes, and examined and cared for the soil and rock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and appropriate laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate. Classification testing (water content, Atterberg Limits and grain size distribution) was carried out on selected soil samples. Point load testing was performed on samples of the the rock core.

The borehole locations were marked in the field with reference to existing land marks such as hydro poles, the existing bridge and the railway track. The ground surface elevations at the borehole locations were established using a local benchmark (Railway track El. 300.7 m) and are considered accurate to within ± 0.1 m. The Elevation of the local benchmark used at site was provided to Golder by Morrison Hershfield in September 2003. The borehole elevations and northing and easting coordinates based on the field measurements are indicated on the Record of Borehole sheets and the borehole locations are shown on the attached Drawing 1. Due to the site topography and difficulty in obtaining accurate measurements, the borehole locations should be considered to be accurate to within ± 0.5 m.

The 1962 boreholes were advanced to depths of 6.7 m to 14.6 m (including rock coring where applicable). No piezometers were installed in these boreholes. The approximate locations of these boreholes, labelled as 62-1 through 62-4, are shown on Drawing 1. The locations of the boreholes are estimated from the drawing provided in the original 1962 report and may differ slightly from the actual location.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Geology

From published geologic information, the site is located in the physiographic region known as the Flamborough plain. The Flamborough plain is a flat to undulating limestone plain from which glaciers have stripped most of the overburden. What little overburden remains overlying the bedrock, apart from localized drumlins, is either bouldery glacial till or sand and gravel (Chapman and Putnam, "The Physiography of Southern Ontario", 3rd Edition, 1984).

The original ground surface at the CP Rail overhead bridge site is at approximately Elevation 300 m. The native overburden at the site consists predominantly of non-cohesive till that varies in composition from silt and sand to sand and gravel. The overburden at the site is underlain by dolostone bedrock from the Amabel Formation at relatively shallow depth; typically varying between 5 m and 6 m below original ground surface.

4.2 Subsurface Conditions and General Overview

The detailed subsurface soil 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 and in Appendix A following the text of this report. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

The locations of the boreholes along with the interpreted stratigraphy profile and sections are shown on Drawings 1 and 2.

In general, the subsoils at the site consist of surficial layers of topsoil and/or fill/reworked native soils placed during construction of the existing road embankments. The fill/reworked soils typically are comprised of silty sand and gravel containing varying amounts of organics, cinders, clay, cobbles and boulders. In BH03-3, BH03-4, and BH03-5 the surficial soils are underlain by a 0.6 m thick brown silt and sand unit. Below this unit and directly underlying fill materials in

BH03-1, BH03-2, and BH03-6 is a gravelly silt and sand to silty sand and gravel till containing cobbles and boulders. The till deposit is underlain by strong dolostone bedrock. The upper portion of the bedrock was found to be moderately to highly weathered with silt and sand found within noticeably saturated fractures. The total overburden thickness was relatively consistent, ranging from 6.1 m at the proposed north and south approach embankments to about 7.6 m below the ground surface in BH03-3 (located on the lower portion of the existing embankment slope at the proposed south abutment). A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil

Topsoil was generally encountered at the existing ground surface at the borehole locations. The thickness of the topsoil ranged from about 0.1 m to 0.9 m. The topsoil contained varied amounts of clay, sand, and gravel.

Laboratory testing carried out on one sample of the topsoil indicated that the natural water content of the sample tested was approximately 25 percent, expressed as a percentage of the dry weight of the soil.

4.2.2 Fill and Reworked Soil

Fill materials and/or reworked soils were encountered at ground surface or directly underlying the topsoil in Boreholes 03-1 to 03-6. Typically, the fill/reworked soil consists of silty sand and gravel containing varying amounts of organics, clay, cinders, cobbles and boulder. The fill/reworked soils extended to depths of between approximately 0.9 m to 2.3 m below ground surface at the boreholes drilled during the current investigation.

Boreholes 62-1 and 62-4, drilled during the 1962 investigation, were drilled through the existing highway embankment. These boreholes encountered up to 8.2 m of fill described as sand and gravel containing boulders. Boreholes 62-2 and 62-3, drilled during the 1962 investigation outside of the area of highway embankment, encountered 0.3 m to 0.8 m of fill materials containing cinders.

Standard Penetration Testing (SPT) 'N' values measured within the fill during the current investigation ranged between 3 and 32 blows per 0.3 m of penetration, indicating the fill materials are very loose to dense.

The natural water content measured on samples of the fill/reworked soils ranged from 9.4 to 12.2 percent.

4.2.3 Silt and Sand

A relatively thin deposit of brown silt and sand was encountered below the fill and reworked soils in boreholes BH03-3, BH03-4, and BH03-5. The top of the deposit was encountered between Elevations 298.8 and 299.0 m. The deposit ranged in thickness from 0.6 m to 0.7 m. Boreholes 62-2 and 62-3 encountered a silty sand deposit that is inferred to be the same deposit. A grain size distribution curve for a sample of this deposit is shown on Figure A-1 in Appendix A.

The SPT 'N' values measured in this deposit ranged from 4 to 27 blows per 0.3 m of penetration, indicating a loose to compact relative density.

Samples of the silt and sand were noted to be moist to wet. The natural water content was determined to range from 16.5 to 20.1 percent.

4.2.4 Gravelly Silt and Sand to Silty Sand and Gravel (Till)

In all boreholes, a till deposit typically comprised of brown gravelly silt and sand to silty sand and gravel was encountered. Zones of clayey silt with sand till were encountered within the predominantly granular till in some areas. Grinding of the augers was noted throughout the majority of the till deposit suggesting that this deposit contains significant amounts of cobbles and boulders. This deposit was also encountered in all 1962 boreholes and was described as silty sand and gravel with boulders.

In boreholes BH03-3, BH03-4, and BH03-5 the till was encountered beneath the silt and sand deposit at Elevations varying from 298.1 m to 298.4 m. In boreholes BH03-1, BH03-2, and BH03-6 the till was observed to directly underlie the fill materials at approximate Elevations 298.5 m to 299.4 m. The till was found to extend to the bedrock surface in all boreholes, ranging in thickness from 2.6 m in BH03-5 to 5.3 m in BH03-1 (a portion of which may be weathered bedrock).

At the current borehole locations the SPT ‘N’ values measured within the till during the current investigation varied from 2 blows per 0.3 m of penetration to in excess of 50 blows per 0.1 m of penetration but were more typically in the range of 10 to 40 blows per 0.3 m of penetration. Based on the SPT ‘N’ values, the till deposit is very loose to very dense. SPT ‘N’ values measured within the till deposit during the 1962 investigation varied from 11 to 81 blows per 0.3 m of penetration.

The natural water content measured on selected samples of this deposit ranged between 4 percent and 11 percent. Grain size distribution curves for samples of this deposit are shown on Figure A-2 in Appendix A. Atterberg limits testing was carried out on one sample of the till from BH03-5. The test results are displayed on Figure A3 in Appendix A and indicate that sample tested was a silt of low plasticity.

4.2.5 Bedrock

Bedrock was encountered in all of the 2003 boreholes and in a majority of the 1962 boreholes. Bedrock was confirmed by coring in boreholes BH03-2 to BH03-5, 62-2, and 62-4 and inferred from refusal to further auger advance in boreholes BH03-1, BH03-6, and 62-3. Borehole 62-1 indicated auger refusal at an elevation several metres higher than the other boreholes at the site. For this reason, and because of the bouldery soil conditions, it is assumed that auger refusal at this particular borehole was due to the presence of a boulder rather than the bedrock.

The bedrock core samples obtained consist of light brown to light grey, fresh, strong, fine to medium grained dolostone, with occasional vugs and styolitic laminae. The boreholes drilled during the 1962 borehole records described the bedrock as limestone. The majority of the bedding planes and joints were horizontal with single vertical joints noted in BH03-3 and BH03-4 respectively. As noted on the Record of Drillholes, some joints were found to contain thin seams of brown silt and clay, which were also associated with a void in BH03-5.

The elevation and depth of the bedrock surface as encountered in the boreholes is given in the table below:

Borehole	Location	Ground Surface Elevation (m)	Bedrock Depth (m)	Bedrock Surface Elevation (m)
BH03-1	South Approach	300.3	*6.1**	*294.2**
BH03-2	South RSS Wall	300.0	5.8	294.2
BH03-3	South Abutment	301.1	6.2	294.9
BH03-4	North Abutment	300.2	6.2	294.1
BH03-5	North RSS Wall	300.2	4.7	295.5
BH03-6	North Approach	300.5	*6.1**	*294.4**
62-2	North Abutment	300.8	6.1	294.7
62-3	South Abutment	300.6	*6.7**	*293.9**
62-4	South RSS Wall	308.4	13.4	295.0

*Bedrock surface inferred from auger refusal and was not confirmed by coring.

Boreholes BH03-2 to BH03-5 were advanced between 0.8 m and 1.8 m into the bedrock by augering before encountering refusal; split spoon samples were obtained in this portion of the rock. The ability to auger into the upper portion of the bedrock suggests that this portion of the bedrock may be more heavily weathered and/or highly fractured than the remainder of rock mass. The split spoon samples retrieved contained a large proportion of silt and sand as well as weathered dolostone gravel sizes. The sand and silt material was evident within the fractures in this portion of the rock. Refusal to augering was encountered during the current investigation at Elevations 293.2 m to 295.0 m.

The Total Core Recovery (TCR) was between 75 and 100 percent and the Solid Core Recovery (SCR) was between 30 and 100 percent; the lower values are associated with core where vertical factures were noted. The Rock Quality Designation (RQD) measured on the core samples ranged from about 30 to 88 percent, indicating a rock mass of poor to good quality. The initial core runs of boreholes BH03-4 and BH03-5 had the lowest RQDs, averaging about 33 percent. The RQDs of the remaining core runs of BH03-4 and BH03-5 and all core runs of BH03-2 and BH03-3 averaged about 77 percent. In general, based on all of the rock core information available, the rock mass is considered to be of good quality.

Point load strength tests were performed on selected samples of the rock core from Boreholes BH03-2 to BH03-5. Axial and diametral point load strength index values are shown on the Record of Drillhole Sheets. Approximate diametral point load UCS (unconfined compressive strength) values range from 54 MPa to 108 MPa with an average of 75 MPa. The axial point load

UCS values range from 47 MPa to 86 MPa with an average of 65 MPa. Using the Intact Rock Strength Classification table, these values indicate that the rock strength is strong to very strong as measured parallel to bedding planes, and medium strong to very strong as measured perpendicular to bedding planes. Overall, the specimens are considered to be strong and assumed to be relatively isotropic in strength based on the ratio of axial to diametral strength. Point load test results are included in Table A2 in Appendix A.

4.2.6 Groundwater Conditions

Details of the groundwater conditions and water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. Water levels from piezometers installed in BH03-3 and BH03-4 were encountered at about Elevations 294.2 m to 294.3 m as noted in the table below:

Borehole	*Ground Surface Elevation (m)*	*Water Level Depth (m)*	*Water Level Elevation (m)*	*Comments*
BH03-3	301.1	7.6	294.2	Piezometer (Oct.9, 2003)
BH03-4	300.2	5.9	294.3	Piezometer (Oct.9, 2003)

In general, the soil samples recovered from the boreholes were noted to be moist above the weathered dolostone. However, samples obtained in boreholes BH03-4 and BH03-5 were noted to be wet below a depth of approximately 2.5 m to 3 m. Furthermore, the water levels measured in the open boreholes during the 1962 investigation varied between approximately Elevation 298 m and 299.7 m. These observations suggest that perched water conditions may exist above the fine-grained portions of the site soils.

It should be noted that groundwater levels in the area are subject to seasonal fluctuations.

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PART B

FOUNDATION INVESTIGATION AND DESIGN
REPLACEMENT OF THE HIGHWAY 6 SOUTH
CPR OVERHEAD STRUCTURE
SITE NO. 35-366, G.W.P. 416-98-00
MINISTRY OF TRANSPORTATION, ONTARIO

5.0 ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed rehabilitation/replacement of the CPR Overhead Structure along Highway 6 south of Highway 401 based on the information provided to us at this time. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at this site and the borehole information from the 1962 investigation at the site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. In addition, the recommendations are made based on the proposed structure configuration and associated works as provided to us by Morrison Hershfield. The recommendations should be reviewed during detail design. 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.

5.1 General

The existing CPR Overhead bridge is a 3-span structure that carries Highway 6 over a single CPR track. The bridge was originally constructed in 1928 and was widened towards the west in 1967. The original bridge structure is supported on shallow spread footings founded at about Elevation 299.0 m while the widened portion of the bridge is founded on steel H-piles driven to found on the dolostone bedrock.

It is understood that the replacement bridge option has been selected as the preferred alternative in accordance with Stage IIB of the Terms of Reference (TOR). This option involves the construction of a new, single-span bridge structure along a new alignment, the new bridge is proposed on the west side of the existing bridge with the construction of the new bridge carried out in two stages utilizing the existing bridge as a detour. We understand that the new bridge will be a single span structure approximately 16.5 m in length.

The existing highway embankment will be widened towards the west as part of the construction of the new bridge approach embankments. Mechanically-reinforced soil retaining wall systems (retained soil system or RSS walls) are proposed to support the east and west sides of the new bridge approach embankments within 16 m to 17 m of the proposed bridge abutments. The RSS walls will have maximum heights of approximately 9 m to 10 m near the abutments. With increasing distance from the abutments, the base of the RSS walls will step up and the walls will be founded either on the existing embankment fill materials or within the new approach embankments.

5.2 Bridge Foundation Options

The soils at the site consist of near surface deposits of topsoil, fill and loose to compact sandy silt to silty sand. These soils are underlain by a very loose to dense till stratum that is typically comprised of silty sand and gravel to gravelly silt and sand and which contains cobbles and boulders. The till deposit is underlain by medium strong to very strong dolostone bedrock, the surface of which was encountered at about Elevation 294 m to 295.5 m. The water levels observed in the open boreholes during the 1962 investigation varied between approximately Elevation 298 m and 299.7 m. The water levels measured in the piezometers installed in Boreholes BH03-3 and BH03-4 during the current investigation varied between Elevation 294.2 m and 294.3 m.

The surficial topsoil, fill materials and loose to compact sandy silt/silty sand deposit are not considered suitable for the subgrade support of the foundation of the bridge. The bridge abutment foundations may be supported on shallow foundations bearing on the undisturbed till deposits. However, due to the low SPT ‘N’ values recorded in portions of this deposit and the variability in the density of this deposit between boreholes, the abutment foundations would need to be designed with a relatively low design bearing pressure at Serviceability Limit States (SLS) and differential settlements of the foundations may occur.

Alternatively, the abutments may be supported on deep foundations bearing on the dolostone bedrock. Deep foundation alternatives include Steel H-piles driven to found on the bedrock or drilled piers/caissons socketed into the bedrock. Recommendations for spread footings, steel H-pile and drilled shaft foundations for the abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs and risks associated with each of the foundation options is presented in Table 1 following the text of this report.

5.3 Shallow Foundations

5.3.1 Geotechnical Resistance

Spread footings for the bridge abutments may be placed on the till deposit at or below about Elevation 298 m. Spread footings placed on a properly prepared subgrade within undisturbed till at or below the design elevation given above may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 350 kPa, assuming a 2.5 m wide footing.

The settlement of the footings will be dependent on the footing size and configuration, on the applied loads and on the distribution and thickness of any loose zones within the till deposit. The settlement performance of the abutment footings will also be dependent on the timing of the construction of the approach embankments/RSS walls. Due to the variability in the SPT 'N' values measured in the till deposit and the thickness of fill materials to be placed in the area of the foundations during construction of the RSS walls, differential settlement of the bridge abutments may occur. The approach embankments/RSS walls should be constructed prior to construction of the bridge structure to limit settlements. Due to the predominantly granular nature of the subsoils, the majority of this settlement is expected to occur during or shortly after the construction period. For preliminary design purposes, the geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 200 kPa for abutment foundations with widths of 3 m.

Alternatively, consideration could be given to use of a compacted Granular 'A' pad underneath the spread footings for the above noted structure. Very loose till, with an SPT 'N' value of 2 blows was encountered between about 296.5 m and 295.5 m in Borehole 03-4. Also, till with 'N' values as low as 8 blows were encountered at other boreholes across the site at the same elevation. For the Granular 'A' pad to be effective, the very loose material must be sub-excavated from below the footing.

It is understood that with the base of the granular pad at Elevation 295.5 m and the top of the pad (base of spread footing) at Elevation 298.0 m, sides slope will extend at a 1 horizontal to 1 vertical (1H:1V) away from the base of the footing. A temporary soldier pile and lagging wall will be required adjacent to the existing rail tracks as it will not be possible to obtain the required 1H:1V side slopes for the granular pad at the south abutment location. The Granular 'A' pad should be compacted in loose lifts not greater than 200 mm in thickness to 100 percent of the material's Standard Proctor maximum dry density. Special care should be taken to ensure proper compaction adjacent to the shoring wall.

For spread footings placed on a well compacted Granular 'A' pad in this configuration, a geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement of 350 kPa may be used for design. In order limit differential settlement between the abutments, the thickness and elevation of the granular pad should be consistent at both the north and south abutments.

The geotechnical resistances provided herein 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 of the load should be taken into account in accordance with Section 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC) and its Commentary, using the curve for non-cohesive soils.

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ', for cast-in-place concrete footings placed on the undisturbed, properly prepared very loose to dense (generally compact) non-cohesive till subgrade at Elevation 298 m may be taken as 0.5. For footings placed on a granular pad as described above, the coefficient of friction may be taken as 0.5. These represent unfactored values; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.3.3 Frost Protection

Shallow foundations should be provided with a minimum of 1.2 m of soil cover for frost protection purposes.

5.3.4 Construction Considerations

Perched water conditions may be encountered during excavations for the spread footings within the till and some inflow should be expected. The quantity of flow is expected to be nominal and pumping from properly filtered sumps placed at the base of the excavation should provide sufficient groundwater control during foundation excavations. Sumps should be maintained outside of the footing area.

The founding soils will be susceptible to disturbance due to water seepage or ponding. Placement of a mud coat will be required at the base of the excavation for the footing area. Exposure without protection of the mud coat will allow water to soften the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. The mud coat should be placed within four hours after footing inspection.

5.4 Deep Foundations

5.4.1 Steel H-Pile Foundations

Due to the low SPT 'N' values measured within the till deposit in some areas, driven piles terminated within the till deposit are not considered suitable for the support of the bridge abutments. Steel H-piles driven to found on the dolostone bedrock may be used for support of the abutments. The surface of the dolostone bedrock was encountered in the boreholes between Elevation 293.9 m and 294.9 m in the vicinity of the proposed abutments, as noted below:

Foundation Element	Borehole Numbers	Depth to Bedrock	Bedrock Surface Elevation
North abutment	BH03-4, 62-2	6.1 m to 6.2 m	294.1 m to 294.7 m
South abutment	BH03-3, 62-3	6.2 m to 6.7 m	293.9 m to 294.9 m

Due to the variable fractured and weathered nature of the surface of the bedrock, it is anticipated that driven steel H-piles may encounter practical refusal at the bedrock surface or may penetrate up to about 1 m below the surface of the bedrock before encountering practical refusal. Based on the bedrock surface elevations noted above, pile tip levels varying between about Elevations 293 m to 294.5 m are anticipated.

5.4.1.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to practical refusal in the dolostone bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the dolostone bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

Pile installation should be in accordance with SP903S01. For driven piles, consideration must be given to the presence of cobbles and boulders within the glacially-derived soils at the site. Driven piles should be equipped suitable driving points in order to make adequate seating of the pile easier given the relatively short pile length and the hardness of the bedrock. Pre-augering at the pile locations may also be required to allow the piles to be driven through the cobble and boulder-bearing till deposit and the contract should include a non-standard special provision regarding the requirements for pre-augering of the pile locations.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking occurs on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

5.4.1.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, kh, is based on the following equation for granular soils:

$$k_h = \frac{n_h z}{B}$$ where n_H is the constant of horizontal subgrade reaction, as given below;
z is the depth (m); and
b is the pile diameter (m).

The following ranges for the value of nh may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments, and the requirement for lateral support in the case of non-integral abutments.

Soil Unit	n_h
Silt and Sand [Elevation 298 m to 299 m]	2 to 5 MPa/m
Silty Sand and Gravel to Gravelly Silt and Sand (Till) [Below Elevation 298 m]	5 to 15 MPa/m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

5.4.1.3 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.4.2 Drilled Shaft Foundations

Drilled shafts/caissons bearing on the dolostone bedrock may be used for support of the abutments. The upper portion of the bedrock is generally more heavily weathered and fractured than the remainder of the rock mass and, therefore, it is recommended that the drilled shafts/caissons be nominally socketed (i.e. a socket depth of approximately 1 shaft diameter with a minimum socket of at least 1 m) into the bedrock. The surface of the dolostone bedrock was encountered in the boreholes between Elevation 293.9 m and 294.9 m in the vicinity of the proposed abutments.

The dolostone bedrock at the site is medium strong to very strong (corresponding to unconfined compressive strengths typically in the range of 45 MPa to 110 MPa). Formation of socket holes in the bedrock is feasible; however, it will be likely necessary to use rock coring or churn drilling techniques to advance the holes to sufficient depth into the bedrock. It is noted that the stronger layers would make churn drilling slow, and the more thickly-bedded portions of the bedrock may be difficult to remove by coring operations, particularly where large diameter sockets are required.

In addition, the overburden bedrock interface at the site are predominantly granular in nature and the water levels measured in the piezometers are at or near the overburden/bedrock interface. Consequently, a temporary liner will be required to support the holes through the overburden during drilling, installation and concrete placement.

5.4.2.1 Axial Geotechnical Resistance

Drilled shafts/caissons socketed nominally (approximately 1 shaft diameter) into the bedrock should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 5 MPa should be used. Serviceability Limit States resistances do not apply to drilled shafts founded on the dolostone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation areas by the Quality Verification Engineer prior to drilled shaft installation and concreting, to ensure that all loose and/or fractured rock has been removed from the foundation areas.

5.4.2.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the drilled shafts, and the reductions due to group effects, may be determined as per Section 5.4.1.2.

5.4.2.3 Frost Protection

The pile caps should be provided with a minimum of 1.2 m of soil cover for frost protection.

5.5 Retained Soil System (RSS) Walls

A mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material and is supported on a strip footing.

The base of the RSS walls at this site are proposed to be founded below the base of the widened embankment in the vicinity of the abutments and step up into the approach embankment with increasing distance back from the abutments. In this regard, portions of the RSS on the east side of the new abutments would be founded within the existing embankment fill whereas the portions of the walls near the abutments and on the west side of the widened roadway embankment would be founded either on native soils or within the new embankment fill materials. The retaining walls are anticipated to be up to 10 m in height at the abutments and decreasing to about 5 m with increasing distance behind the abutments.

The boreholes drilled during the current investigation were located beyond the toe of the existing embankments and typically encountered surficial deposits of topsoil and/or fill materials that contained significant amounts of organics and/or cinders in some areas. These materials were not encountered in boreholes drilled through the embankment during the 1962 investigation. Any topsoil, organic matter, fill containing deleterious materials (e.g. cinders) and softened / loosened soils should be stripped from below the RSS wall foundations.

5.5.1 Geotechnical Resistance

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall. The footing must be founded below any topsoil, loose fill or unsuitable native soils. For an assumed width of 0.6 m for the facing footing and assuming the footing is placed on properly prepared subgrade, a factored geotechnical resistance at ULS of 150 kPa may be used for design.

Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as two-thirds of the height of the wall, the following factored geotechnical resistances at ULS may be used for assessment of the reinforced mass bearing capacity founded on the properly prepared embankment fill materials or silty sand/sandy silt and silty sand and gravel to gravelly silt and sand till deposits. Allowance should be made for subexcavation and replacement with compacted granular fill if unsuitable soils are encountered at the founding level.

Wall Height	Assumed Width	Factored Geotechnical Resistance at ULS
5 m	3.4 m	200 kPa
9 m	6 m	325 kPa

The settlement of the RSS walls (both the reinforced mass as well as the facing footing) will occur as a result of the loading due to the embankment itself, since the walls are incorporated into the embankment. The geotechnical resistance at SLS, for 25 mm of settlement resulting from the combined RSS wall and embankment loading, may be taken as 200 kPa for portions of the RSS walls founded within the embankment fill behind the abutments.

For the portion of the RSS wall adjacent to the abutment foundations, the settlement performance will be dependent on the type of abutment foundation system utilized. If the bridge is supported on driven piles on bedrock or on spread footings formed on a compacted granular pad (as discussed in Section 5.3.1), the SLS value for the RSS wall foundation for 25 mm of settlement will be greater than the geotechnical resistance at ULS and therefore the ULS value will govern. If the bridge foundations are supported on shallow spread footings founded on the native till material at Elevation 298 m, the settlement of the RSS wall will be influenced by the embankment loading behind the bridge abutments and the bridge loading.

The majority of the settlement of the RSS walls will occur during construction since the founding soils are essentially granular (i.e. granular fill materials, silty sand/sandy silt or silty sand and gravel till to gravelly silt and sand till) and are underlain by bedrock at a relatively shallow depth.

5.5.2 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted granular fill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, tan φ', between the compacted granular fill of the RSS wall and the existing granular fill materials or the native silty sand to sand and gravel soils may be taken as 0.58. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.5.3 Stability

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. In this regard, the internal stability must also be checked for seismic loading.

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed wall height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

Properly designed and constructed RSS walls at this site will have a factor of safety of greater than 1.3 against deep-seated slope instability.

5.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any 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.

The following recommendations are made concerning the design of the walls. It should be noted that 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 Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. 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 3501.00 and 3504.00.

- 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 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06; heavy compaction equipment should not be used within a lateral distance behind the structure equal to the current height of the fill above the base of the structure. 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 in Figure C6.9.1(l) of the Commentary to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the Commentary to the CHBDC).

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

Soil Unit Weight:	21 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 as placed 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	0.31
At rest, K_o	0.43	0.47

- 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.

5.7 Embankment Design and Construction

The existing highway embankment will be widened towards the west as part of the construction of the new bridge approach embankments. The construction of the approach embankments will require the placement of additional fill materials. The thickness of the fill materials is anticipated to vary from less than 1 m near the crest of the existing embankment up to about 10 m at the new crest of the west approach embankment.

5.7.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from the existing embankment side slopes and below the approach embankment areas, and all subgrade soils should be proof-rolled prior to fill placement in accordance with OPSS 206. Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent 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 per cent of the Standard Proctor maximum dry density. 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.

Benching of the existing embankment sideslopes should be carried out to key in the new fill materials. Where the embankment height is greater than 8 m, a mid-height berm at least 2 m in width is required. To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding is recommended.

5.7.2 Approach Embankment Stability

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis, to check that a minimum factor of safety of 1.3 is achieved for the proposed approach embankment height and geometry under static conditions. This minimum factor of safety is considered appropriate for the embankments at this site considering the design requirements and the available field and laboratory testing data.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, approach embankments with maximum heights of up to approximately 9 m and side slopes maintained at 2 horizontal to 1 vertical (2H:1V will have a factor of safety of greater than 1.3 against deep-seated slope instability. Static slope stability analyses for this embankment configuration were carried out using the following parameters, based on field and laboratory test data and accepted correlations:

Soil Deposit	***Bulk Unit Weight***	***Effective Friction Angle***
Existing And New Embankment Fills	22 kN/m^3	32°
Silty Sand to Sandy Silt	20 kN/m^3	30°
Gravelly Silt and Sand/Silty Sand and Gravel (Till)	22 kN/m^3	32°

The liquefaction potential of the soils below the embankment under seismic loading has been considered using the empirical method outlined in Section C.4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, a factor of safety of greater than 1.1 against liquefaction for an earthquake of magnitude 7.5 is obtained for the native site soils below the water table.

Pseudo-static methods of slope stability analysis indicate that the yield acceleration required to reduce the factor of safety against slope instability to 1.0 is less than the peak ground acceleration for the design earthquake event. Therefore, significant embankment deformations are not anticipated as a result of the design earthquake event.

5.7.3 Approach Embankment Settlement

Settlement of the approach embankments will occur due to compression of the new embankment fill itself, as well as compression of the existing embankment fill materials and cohesionless overburden soils that underlie the area of embankment widening.

Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude, since the majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fill materials would occur after construction.

The settlement of the embankments as a consequence of compression of the existing embankment fill materials and underlying overburden soils is expected to be less than about 25 mm. This maximum settlement will occur under the crest of the new widened embankment and represents the total differential settlement with respect to the existing embankment; however, the differential will be gradual due to the configuration of the widening and the anticipated pressure bulb. In order to minimize differential settlement between the existing and widened portion of the embankment, the newly placed embankment fill should be keyed into the existing embankment as per OPSD 208.01.

5.8 Design and Construction Considerations

5.8.1 Excavation

Excavations required for the construction of spread foundations or pile caps would extend to about 2 m to 3 m depth below the existing ground surface. The excavations will typically extend through up to approximately 2 m of variable fill materials overlying less than 1 m of loose to compact silt and sand. These materials are in turn underlain by a very loose to dense silty sand and gravel to gravelly silt and sand till.

The groundwater level measured in the piezometers installed in Boreholes BH03-3 and BH03-4 was at an elevation of approximately 292.4 m (i.e. about 6 m to 7.5 m below ground surface). However, the water levels measured in the open boreholes during the 1962 investigation varied between approximately Elevation 298 m and 299.7 m and samples collected from within the till unit at BH03-4 and BH03-5 were observed to be wet below approximately 2.5 m depth (Elevation 297.5 m). These observations suggest that perched water conditions may exist above the fine-grained portions of the site soils.

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 existing fill materials and the loose to compact silt and sand deposit are classified as Type 3 soil according to OHSA. The silty sand and gravel to gravelly silt and sand till deposit is classified as a Type 2 to Type 3 soils according to OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through the existing fill materials and overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) assuming that appropriate groundwater control is carried out.

5.8.2 Groundwater and Surface Water Control

Seepage from zones of perched water within the fill materials and native soils should be expected particularly where sandy zones are intercepted in the excavation. The seepage through these deposits is expected to be minor and pumping from well-filtered sumps located at the base of the excavation within the glacial till should provide adequate groundwater control during foundation excavations.

As noted in Section 5.4.2, if drilled shafts are adopted at this site, the use of a temporary liner will be required within the overburden to support the auger holes during pile or concrete placement.

5.8.3 Excavation Support

The new bridge structure is proposed to be constructed in stages utilizing the existing bridge as a detour structure. Temporary roadway protection is anticipated to be required as space restrictions are not anticipated to permit excavations for the abutment foundations/pile caps and the RSS walls to be made within open cut. Based on the subsurface conditions at the site and the likely excavation geometry, it is anticipated that a soldier pile and lagging system using anchors or rakers to provide lateral support would be suitable. The lagging should be wrapped with filter cloth to prevent loss of fines in areas where the temporary shoring intercepts zones of perched water conditions.

Support to the soldier pile and lagging walls may be provided by anchors or rakers. The raker / anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

The temporary excavation support system should be designed and constructed in accordance with MTO’s Special Provision 539S01. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP 539S01.

Obstructions

The native soils at the site are glacially-derived and, as such, are expected to contain cobbles and boulders. Indeed, the presence of cobbles and/or boulders was inferred from grinding of the augers during borehole advance, and numerous cobbles were recovered during augering.

The presence of such obstructions will affect the installation of driven steel H-piles or drilled shaft foundations. Ultimately, provision will have to be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

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TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES

Footing Option	*NF*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread Footings on Loose to Compact Silty Sand	X	Minimal excavation. Low impact on existing structures.	Very low geotechnical resistance at Serviceability Limit States.	Less expensive than the deeper spread footing options.	Differential settlement anticipated between abutments.
Spread Footings on Gravelly Silt and Sand to Silty Sand and Gravel (Till) – Below Elevation of Approximately 298 m		Minimal excavation. Low impact on existing structures	Geotechnical resistance at Serviceability Limit States is low. Magnitude of foundation settlements affected by construction of RSS walls/approach embankment.	Less expensive than deep foundation options.	Some differential settlement anticipated between abutment foundations due to variability in till deposit and thickness of fill materials to be placed.
Spread Footings on Bedrock	X	Increased capacity over spread footings on overburden. Negligible differential settlement.	Major excavation required with difficulties anticipated excavating through material containing boulders. Groundwater control and temporary shoring likely required.	Increased cost of deep excavation as well as temporary shoring and groundwater control costs.	Excavation for new bridge foundations requires temporary excavation support to protect existing structure/embankments.
Piles driven into Silty Sand and Gravel (Till)	X	Minimized hard driving through bouldery deposit till.	Lower capacity than piles on bedrock.	Pile foundations may be more expensive than spread footings.	Piles which meet refusal on cobbles and/or boulders in till deposit may be underlain by loose portion of till resulting in unacceptable settlements.
Piles driven to Dolostone Bedrock		Differential settlement between abutment foundations minimized. Increased design capacity with respect to piles within silty sand and gravel till.	Difficulties anticipated driving through bouldery till deposit; may be excessive vibrations on existing bridge. Predrilling may be required.	Increased pile length but with higher capacity may reduce number of piles; there will be costs associated with predrilling.	Vibration during driving could have adverse affect on existing structure; may require predrilling to ensure pile penetrate to bedrock as well as to minimize vibration.
Caissons on/in Dolostone Bedrock		Differential settlement minimized. Higher bearing capacities than spread footings on native soils.	Difficult to auger through bouldery till deposit. Temporary liners required for groundwater control. Socketing into bedrock may be difficult.	Cost may be higher than driven piles due to requirements for liners. Increased caisson length but higher capacity.	Difficulty may be encountered socketing liner into dolostone bedrock to seal off water; downhole inspection may not be possible.

NF: Not considered a feasible founding alternative for this project

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	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.

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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
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_1	liquid limit
w_p	plastic limit
l_p	plasticity index = $(w_1 - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_C	consistency index = $(w_1 - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (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 δ
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
* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding
FO - Foliation/Schistosity
CL - Cleavage
SH - Shear Plane/Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
MF - Mechanical Fracture
$\parallel$ - Parallel To
$\perp$ - Perpendicular To

P - Polished
S - Slickensided
SM - Smooth
R - Ridged/Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

Golder Associates

PROJECT 03-1111-005

RECORD OF BOREHOLE No BH03-1

METRIC

W.P. 416-98-00 LOCATION N 4810242.4; E 256840.9 ORIGINATED BY CG

DIST HWY 6 BOREHOLE TYPE 108mm I.D. POWER AUGER COMPILED BY KG

DATUM Geodetic DATE October 1, 2003 CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
300.3	GROUND SURFACE									
0.0	Topsoil									
0.1	Silty SAND and GRAVEL, some organics Compact Brown to dark brown Moist (FILL)	1	SS	22	300					
299.4 0.9	Gravelly SILT and SAND to Silty SAND and GRAVEL, trace clay, contains cobbles and boulders Compact to very dense Brown Moist (TILL)	2	SS	18		o				
		3	SS	14	299	o				
		4	SS	41	298					
		5	SS	20	297	o	39	32	25	4
		6	SS	50/0.10	296					
		7	SS	50/0.08		o				
	Possibly fractured rock below 5.2m depth				295					
294.2 6.2	End of Borehole	8	SS	50/0.05						

Columns also shown: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%).

Notes:
1. Auger and spoon refusal at 6.2m depth (Elev. 294.2m)
2. Augers grinding with advancement from about 1.5m depth to refusal
3. Open hole dry on completion of drilling

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

Golder Associates

RECORD OF BOREHOLE No BH03-2

METRIC

PROJECT 03-1111-005			
W.P. 416-98-00	LOCATION N 4810251.9; E 256824.1	ORIGINATED BY CG
DIST	HWY 6	BOREHOLE TYPE 108mm I.D. POWER AUGER	COMPILED BY KG
DATUM Geodetic		DATE September 30, 2003	CHECKED BY SEP

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
300.0	GROUND SURFACE
0.0	Topsoil
0.2	SAND, some silt and organics, trace to some gravel, trace clay and coal, contains boulders and cobbles Loose to compact Dark brown to brown Moist (FILL)
298.5 / 1.6	Gravelly SILT and SAND to Silty SAND and GRAVEL, trace clay, contains cobbles and boulders Compact to very dense Brown Moist (TILL)
294.2 / 5.8	DOLOSTONE (BEDROCK) Silt and sand filled fractures Moderately to highly weathered Close to very closely fractured Weak Light Brown Wet
293.4 / 6.7	DOLOSTONE (BEDROCK) with occasional vugs and stylolitic laminae Fresh Strong Fine to medium grained Light brown to light grey Bedrock cored from 6.7m to 9.6m depth For Bedrock coring details see Record of Drillhole BH03-2
290.4 / 9.6	End of Borehole

Notes:

1. Auger and spoon refusal at 6.7m depth (Elev. 293.4m)
2. Augers grinding with advancement from about 1.5m depth to refusal
3. Water introduced for coring operations; water level in open hole on completion not relevant

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	12	
3	SS	14	
4	SS	14	
5	SS	28	
6	SS	33	28 35 31 6
7	SS	64	
8	SS	50/0.08	

ELEVATION SCALE: 299, 298, 297, 296, 295, 294, 293, 292, 291

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

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

RECORD OF DRILLHOLE: BH03-2

PROJECT: 03-1111-005

LOCATION: N 4810251.9; E 256824.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: September 30, 2003

DRILL RIG: Track Mounted CME55

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

DATUM: Geodetic

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page														
7	NQ CORING	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae Fresh Strong Fine to medium grained Light brown to light grey (AMABEL FORMATION) Very close to moderately closely fractured Majority of joints are horizontal Upper 0.1m of Run No.1 consists of rock fragments		293.30 6.70	1		0									
8					2		100									
9				290.40 9.60	3		100									
		End of Drillhole														
10																
11																
12																
13																
14																
15																
16																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CG

CHECKED: SEP

Golder Associates

RECORD OF BOREHOLE No BH03-3 — 1 OF 1 — **METRIC**

PROJECT 03-1111-005
W.P. 416-98-00 — LOCATION N 4810262.2; E 256817.5 — ORIGINATED BY CG
DIST ___ HWY 6 — BOREHOLE TYPE 108mm I.D. POWER AUGER — COMPILED BY KG
DATUM Geodetic — DATE September 29,2003 — CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
301.1	GROUND SURFACE						
0.0	Topsoil				301		
0.2	Silty SAND and GRAVEL, some organics, trace clay Loose to dense Brown Moist (FILL)	1	SS	4			
		2	SS	32	300		
299.7 1.4	Silty SAND and GRAVEL Compact Brown Dry (FILL)	3	SS	26			
298.8 2.3	SILT and SAND, trace clay Compact Brown Moist	4	SS	27	299		0 44 49 7
298.2 2.9	Gravelly SILT and SAND to Silty SAND and GRAVEL, trace clay, contains boulders and cobbles Compact to dense Brown Moist (TILL)	5	SS	16	298		
		6	SS	26	297		
		7	SS	35	296		
294.9 6.2	DOLOSTONE (BEDROCK) Silt and sand filled fractures Moderately to highly weathered Close to very closely fractured Weak Light brown Wet	8	SS	57	295		
293.5 7.6	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae Fresh Strong Fine to medium grained Light brown to light grey Bedrock cored from 7.6m to 10.6m depth For Bedrock coring details see Record of Drillhole BH03-3	~~9~~	~~SS~~	~~50/0~~	294 293 292 291		
290.5 10.6	End of Borehole Notes: 1. Auger and spoon refusal at 7.6m depth (Elev. 293.5m) 2. Augers grinding with advancement from about 1.5m depth to refusal 3. Water level in piezometer measured at 6.9m (Elev. 294.2m) below ground surface on Oct. 9, 2003						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED — 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity — O $^{3\%}$ STRAIN AT FAILURE

MISS_MTO 03-1111-005HWY6.GPJ ON_MOT.GDT 30/1/04

PROJECT: 03-1111-005

RECORD OF DRILLHOLE: BH03-3

SHEET 1 OF 1

LOCATION: N 4810262.2; E 256817.5

DRILLING DATE: September 30, 2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track Mounted CME55

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH: COLOUR / % RETURN	RECOVERY: TOTAL CORE % (80 60 40 20)	RECOVERY: SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page		293.50												
8	NQ CORING	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae Fresh Strong Fine to medium grained Light brown to light grey (AMABEL FORMATION) Majority of joints are horizontal. One 0.1m long vertical fracture was noted in Run No.1 Thin (<5mm) brown clay seams were noted at 7.1m and 7.6m depths		7.60	1		0									
9					2		0									
10				290.50 10.60	3		0									
		End of Drillhole														
11																
12																
13																
14																
15																
16																
17																

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CG

CHECKED: SEP

PROJECT 03-1111-005

RECORD OF BOREHOLE No BH03-4

METRIC

W.P. 416-98-00	LOCATION N 4810265.1; E 256804.3		ORIGINATED BY CG
DIST	HWY 6	BOREHOLE TYPE 108mm I.D. POWER AUGER	COMPILED BY KG
DATUM Geodetic	DATE September 29,2003		CHECKED BY SEP

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10, 20, 30 / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
300.2	GROUND SURFACE							
0.0	SAND, some rootlets, gravel, cinders/coal, occasional grey sandy silt pockets; Loose; Black; Moist; (FILL)	1	SS	5				
299.6 / 0.6	(continued)	2	SS	15				
299.0 / 1.2	Sandy SILT to Silty SAND, trace to some clay, trace gravel, some organics; Compact; Brown; Moist; (FILL)	3	SS	11				
298.4 / 1.8	SILT and SAND, trace clay, trace gravel; Compact; Brown; Moist	4	SS	38	65	22	14	2
	Gravelly SILT and SAND to Sandy GRAVEL, trace to some clay, contains boulders and cobbles; Loose to dense; Brown; Moist to Wet; (TILL)	5	SS	11				
		6	SS	2				
	Contains occasional zones of clayey silt with sand (TILL)	7	SS	32				
294.1 / 6.2	DOLOSTONE (BEDROCK); Silt and sand filled fractures; Moderately to highly weathered; Close to very closely fractured; Weak; Light brown; Wet	8	SS	50/0.13				
293.2 / 7.0	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae; Fresh; Strong; Fine to medium grained; Light brown to light grey; Bedrock cored from 7.0m to 10.2m depth; For Bedrock coring details see Record of Drillhole BH03-4							
290.0 / 10.2	End of Borehole							

Notes:
1. Auger and spoon refusal at 7.0m depth (Elev. 293.2m)
2. Augers grinding with advancement from about 2.4m depth to refusal
3. Water level in piezometer measured at 5.9m (Elev. 294.3m) below ground surface on Oct. 9, 2003

Elevation scale: 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290

$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

RECORD OF DRILLHOLE: BH03-4

PROJECT: 03-1111-005

SHEET 1 OF 1

LOCATION: N 4810265.1; E 256804.3

DRILLING DATE: September 29,2003

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Track Mounted CME55

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER 0.3 (5 10 15 20)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	DIAMETRAL POINT LOAD INDEX (MPa) (2 4 6)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page														
7	NQ CORING	DOLOSTONE (BEDROCK) with occasional vugs and stylolitic laminae Fresh Strong Fine to medium grained Light brown to light grey (AMABEL FORMATION) Majority of joints are horizontal. A vertical joint was observed from 7.1 to 7.6m depth.		293.20 7.00	1		0									
8					2		100									
9					3		100									
10				290.00 10.20												
		End of Drillhole														

11

12

13

14

15

16

17

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: CG

CHECKED: SEP

Golder Associates

RECORD OF BOREHOLE No BH03-5 1 OF 1 **METRIC**

PROJECT 03-1111-005
W.P. 416-98-00 LOCATION N 4810266.8; E 256784.8 ORIGINATED BY CG
DIST HWY 6 BOREHOLE TYPE 108mm I.D. POWER AUGER COMPILED BY KG
DATUM Geodetic DATE October 1, 2003 CHECKED BY SEP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.2	GROUND SURFACE					
0.0 / 299.9	Topsoil	1	SS	4	300	
0.3	SAND, some silt and organics, trace to some gravel Loose Brown to dark brown Moist (FILL)	2	SS	3		
298.8 / 1.4	SILT and SAND, trace clay Loose Brown Moist	3	SS	4	299	
298.1 / 2.1	Gravelly SILT and SAND to Silty SAND and GRAVEL, trace clay, contains boulders and cobbles with depth Loose to compact Brown Moist to wet (TILL) Contains occasional zones of clayey silt with sand (TILL)	4	SS	8	298	
		5	SS	13	297	38 32 36 4
		6	SS	11	296	
295.5 / 4.7	DOLOSTONE (BEDROCK) Silt and sand filled fractures Moderately to highly weathered Close to very closely fractured Weak Light brown Wet	7	SS	38	295	
		8	SS	50/0.13	294	
293.8 / 6.5	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae Fresh Strong Fine to medium grained Light brown to light grey Bedrock cored from 6.4m to 9.4m depth For Bedrock coring details see Record of Drillhole BH03-5				293 292 291	
290.9 / 9.3	End of Borehole Notes: 1. Auger and spoon refusalt at 6.2m depth (Elev. 294.0m) 2. Augers grinding with advancement from about 3.0m depth to refusal 3. Water introduced for coring operations; water level in open hole on completion not relevant					

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT

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

PROJECT: 03-1111-005

RECORD OF DRILLHOLE: BH03-5

SHEET 1 OF 1

LOCATION: N 4810266.8; E 256784.8

DRILLING DATE: October 1, 2003

DATUM: Geodetic

DRILL RIG: Track Mounted CME55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Geo-Environmental Drilling Ltd.

FR/FX-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. / DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH: COLOUR / % RETURN	RECOVERY: TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA: DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Refer to previous page													
7–9	NQ CORING	DOLOSTONE (BEDROCK) with occasional vugs and styolitic laminae Fresh Strong Fine to medium grained Light brown to light grey (AMABEL FORMATION) Upper 0.15m of Run No.1 consists of rock fragments Thin (<5mm) brown clay seams were noted at 7.1m and 7.6m depths At 7.6m, core barrel dropped 5-8cm, indicating a void.		293.70 6.50	1 2		0 0								
		End of Drillhole		290.90 9.30											

DEPTH SCALE 1 : 50

LOGGED: CG

CHECKED: SEP

MISS_ROCK 03-1111-005-ROCK.GPJ GLDR_CAN.GDT 30/1/04

Golder Associates

PROJECT 03-1111-005 | RECORD OF BOREHOLE No BH03-6 | 1 OF 1 | METRIC

W.P. 416-98-00 | LOCATION N 4810280.5; E 256767.3 | ORIGINATED BY CG

DIST | HWY 6 | BOREHOLE TYPE 108mm I.D. POWER AUGER | COMPILED BY KG

DATUM Geodetic | DATE October 1, 2003 | CHECKED BY SEP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
300.5	GROUND SURFACE						
0.0	Topsoil, some sand and clay, trace gravel, occasional sandy pockets Loose Black to brown Moist	1	SS	8	300	~25	
299.6 0.9	Silty SAND and GRAVEL Compact Brown Moist (FILL)	2	SS	16			
299.1 1.4	Gravelly SILT and SAND to Silty SAND and GRAVEL, trace clay, contains boulders and cobbles Compact to very dense Brown Moist (TILL)	3	SS	23	299	~3	24 41 31 4
		4	SS	23	298		
		5	SS	74	297	~6	
		6	SS	59			
		~~7~~	~~SS~~	~~50/0.05~~	296	~4	
	Possibly fractured rock below 5.2m depth				295		
294.4 6.1	End of Borehole	~~8~~	~~SS~~	~~50/0~~			

Notes:
1. Auger and spoon refusal at 6.1m depth (Elev. 294.4m)
2. Auger grinding with advancement from about 1.8m depth to refusal
3. Open hole dry on completion of drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

$+^3, \times^3$: Numbers refer to Sensitivity   O $^{3\%}$ STRAIN AT FAILURE

DIST. 4 HWY. No. 6
CONT No.
WP No.416-98-00

SHEET

CPR OVERHEAD
HWY No.6 AT PUSLINCH

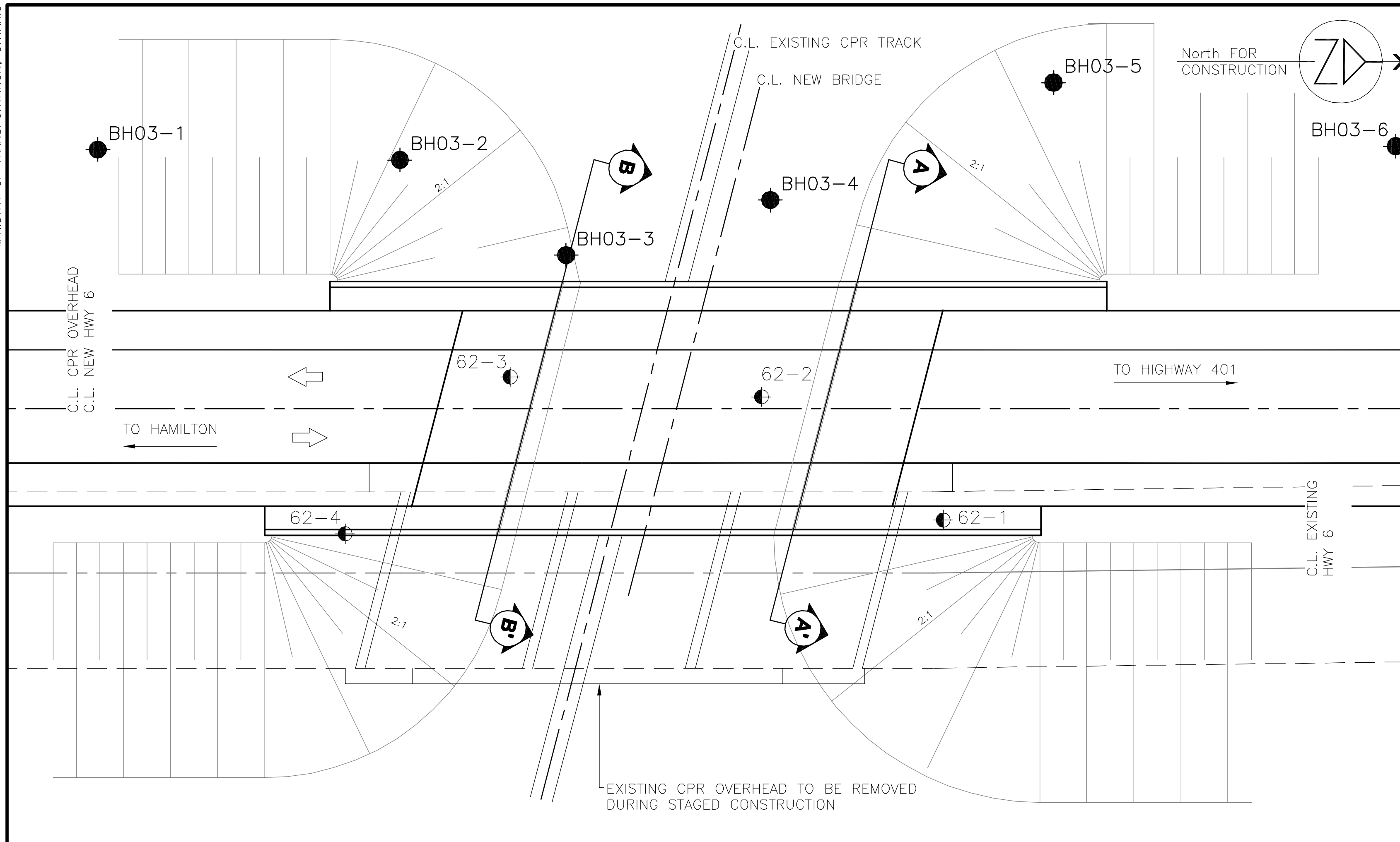
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

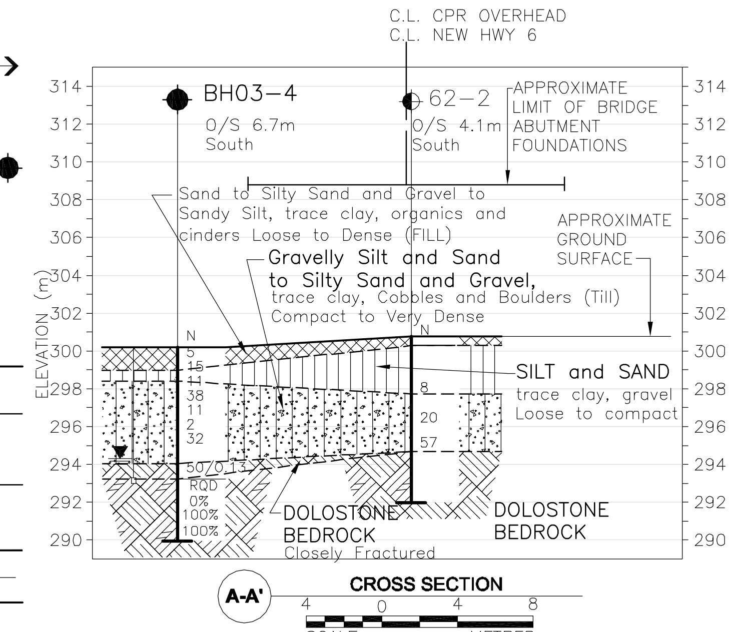
METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



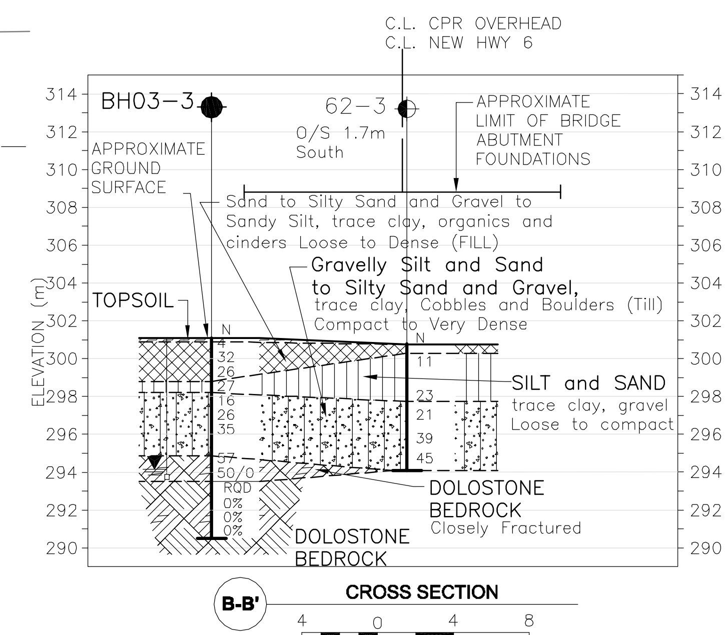
PLAN

SCALE 4 0 4 8 METRES



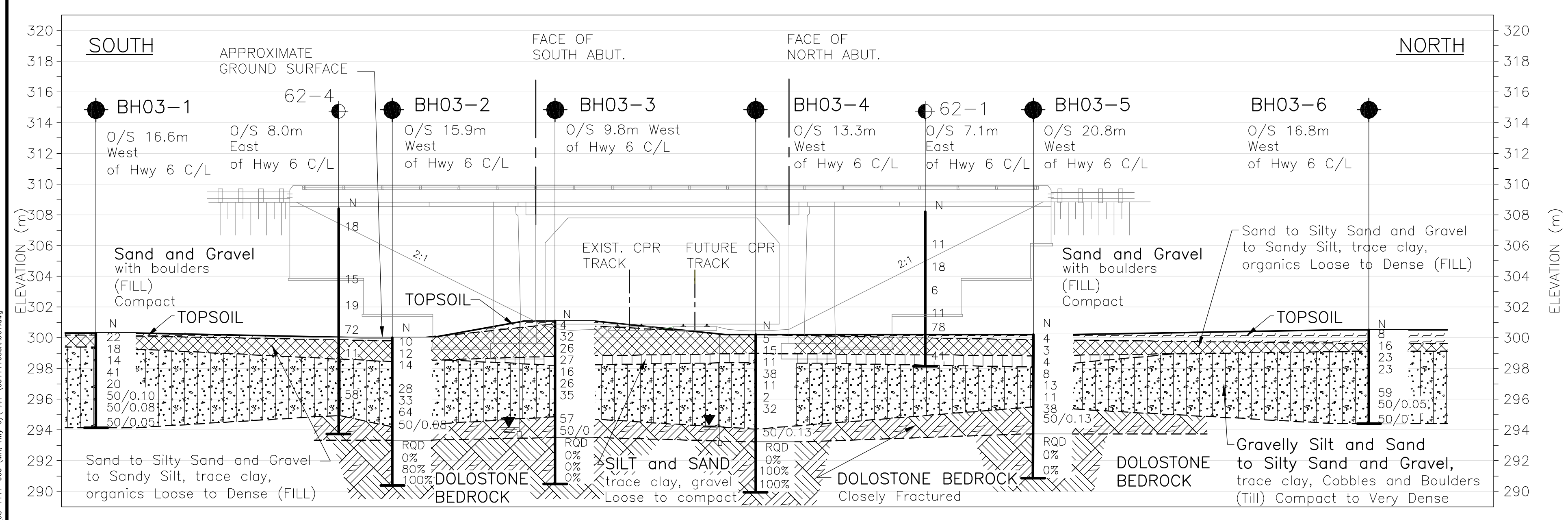
A-A' CROSS SECTION

SCALE 4 0 4 8 METRES



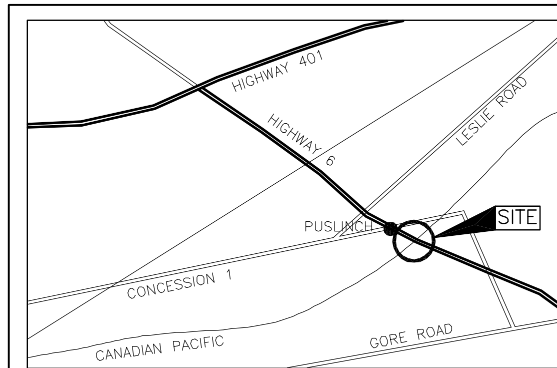
B-B' CROSS SECTION

SCALE 4 0 4 8 METRES



CENTRELINE PROFILE

SCALE 4 0 4 8 METRES



KEY PLAN

1km 0 1km

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)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on October 9, 2003
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES NORTHING	CO-ORDINATES EASTING
BH03-1	300.3	4810242.4	256840.9
BH03-2	300.0	4810251.9	256824.1
BH03-3	301.1	4810262.2	256817.5
BH03-4	300.2	4810265.1	256804.3
BH03-5	300.2	4810266.8	256784.8
BH03-6	300.5	4810280.5	256767.3
62-1	308.2	4810288.3	256804.0
62-2	300.8	4810276.0	256810.6
62-3	300.6	4810267.5	256824.3
62-4	308.4	4810271.5	256838.2

NOTES

1. The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

2. Location of Boreholes 62-1 to 62-4 are approximate only and are based on plans provided in Foundation Investigation Report, widening of Existing (or Completely New) Overhead, Hwy No. 6 and C.P.R. at the village of Puslinch, W.J. 62-F-107, W.P. 75-61, Geocres no. 40P8-29 dated October 1962

REFERENCE

Preliminary General Arrangement Drawing provided in digital format by Morrison Hershfield, drawing file no. 1016-S01.dwg, received September 10, 2003.

NO.	DATE	BY	REVISION

Geocres No. 40P8-131

HWY. 6	PROJECT NO. 03-1111-005		DIST. 4
SUBM'D. SEP	CHKD. SEP	DATE: JAN. 2004	SITE: 35-366
DRAWN: JFC	CHKD. SEP	APPD.	DWG. 1

PLOT DATE: January 30, 2004
FILENAME: T:\Projects\2003\03-1111-005 (MH, Hwy 6)\-AA-\0311110050A001.dwg

APPENDIX A

LABORATORY TEST DATA

TABLE A1

SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBEF 031-111005
PROJECT NAME MH / CPR Overhead Structure / Hwy 6
DATE TESTED October, 2003

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
1	2	2.5-4.0	0.76-1.22	3.0%	
1	3	5.0-7.5	1.52-2.29	5.9%	
1	5	10.0-11.5	3.05-3.51	6.2%	
1	7	15.0-16.5	4.57-5.03	3.9%	
2	2	2.5-4.0	0.76-1.22	12.2%	Organics
2	4	7.5-9.0	2.29-2.74	8.1%	
2	6	12.5-14.0	3.81-4.27	7.6%	
2	8	20.0-21.5	6.10-6.55	8.6%	
3	2	2.5-4.0	0.76-1.22	10.3%	
3	4	7.5-9.0	2.29-2.74	20.1%	
3	7	15.0-16.5	4.57-5.03	5.1%	
3	8	20.0-21.5	6.10-6.55	6.1%	
4	2	2.5-4.0	0.76-1.22	9.4%	Organics
4	3	5.0-6.5	1.52-1.98	19.5%	
4	4	7.5-9.0	2.29-2.74	4.3%	
4	6	12.5-14.0	3.81-4.27	11.0%	
5	2	2.5-4.0	0.76-1.22	11.9%	
5	3	5.0-6.5	1.52-1.98	16.5%	
5	5	10.0-11.5	3.05-3.51	9.0%	
5	6	12.5-14.0	3.81-4.27	11.1%	LL=15.3, PL=11.7, PI=3.6
6	1	0.0-1.5	0.00-0.46	25.0%	Organics
6	3	5.0-6.5	1.52-1.98	3.0%	
6	5	10.0-11.5	3.05-3.51	6.2%	
6	7	15.0-16.5	4.57-5.03	4.3%	

TABLE A2 - POINT LOAD TESTS ON ROCK SAMPLES

PROJECT NO. 03-1111-005

TITLE Highway 6 CPR Overhead Stucture

DATE Oct. 9, 2003

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core [2] Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	**Is (50mm) (MPa)**	**Approx. [1] UCS (MPa)**
BH03-2	1	7.5	A	60.00	47.60	60.30	5,000	6.78	1.863		2.027	47
	2	7.5	D		47.60		5,000	6.78		2.990	2.925	67
	3	8.4	A	57.00	47.60	58.78	7,000	9.49	2.746		2.953	68
	4	8.4	D		47.60		5,000	6.78		2.990	2.925	67
BH03-3	1	8.2	D		47.60		7,000	9.49		4.186	4.095	94
	2	8.2	A	72.00	47.60	66.06	8,000	10.84	2.484		2.816	65
BH03-4	1	7.9	D		47.60		6,000	8.13		3.588	3.510	81
	2	9.4	A	65.00	47.60	62.76	6,000	8.13	2.064		2.286	53
	3	9.4	D		47.60		4,000	5.42		2.392	2.340	54
BH-05	1	7.5	D		47.60		4,000	5.42		2.392	2.340	54
	2	7.9	A	50.00	47.60	55.05	8,000	10.84	3.577		3.736	86
	3	7.9	D		47.60		6,000	8.13		3.588	3.510	81
	4	8.5	A	60.00	47.60	60.30	8,000	10.84	2.981		3.243	75
	5	8.5	D		47.60		5,000	6.78		2.990	2.925	67
	6	8.5	D		47.60		8,000	10.84		4.784	4.680	108

Overall Mean	**71**
Mean Axial	65
Mean Diametral	75
Anisotrophy Ratio (D/A)	1.1

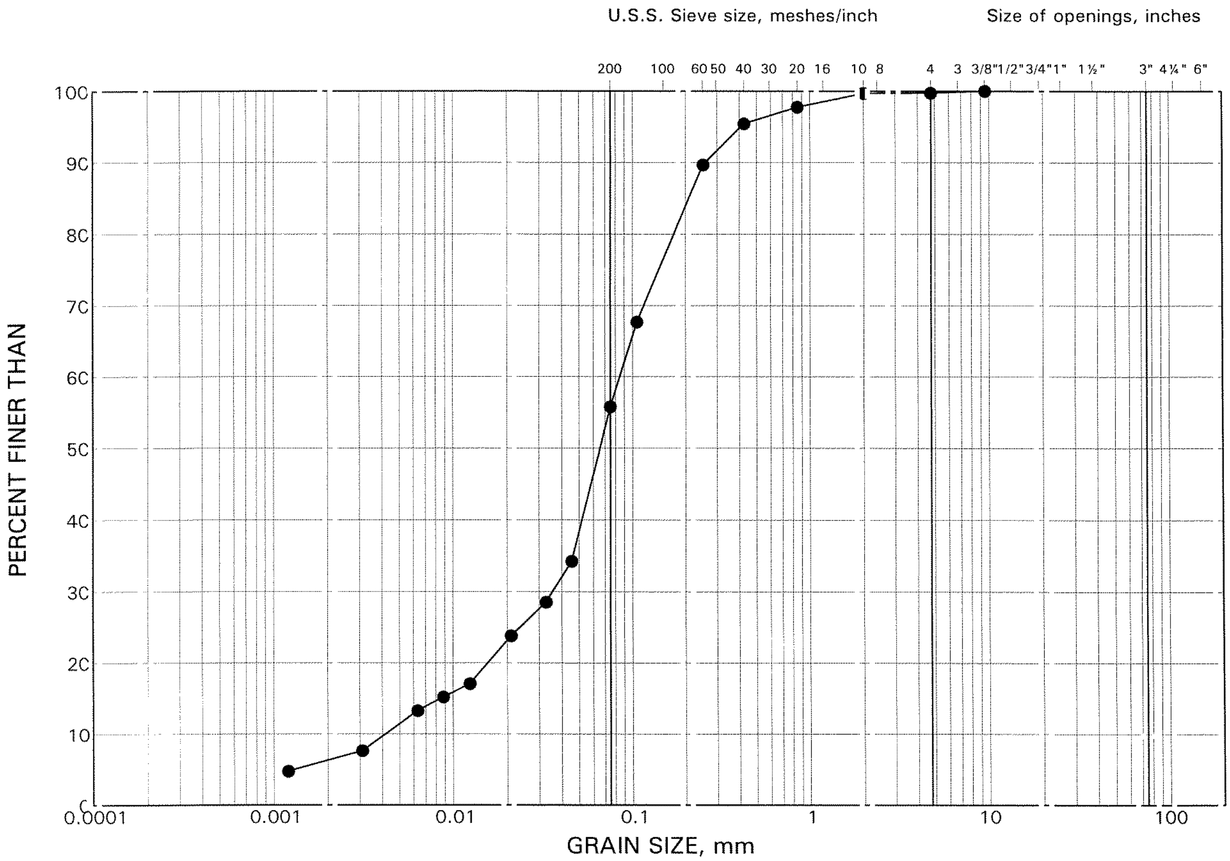
These DOLOSTONE specimences are STRONG ROCK and appear to be relatively isotropic in strength based on Axial and Diametral test

(1) Is_{50} x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-6(

(2) Actual distance between point load cones at time of failure.

GRAIN SIZE DISTRIBUTION
Silt and Sand

FIGURE A-1



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

LEGEND

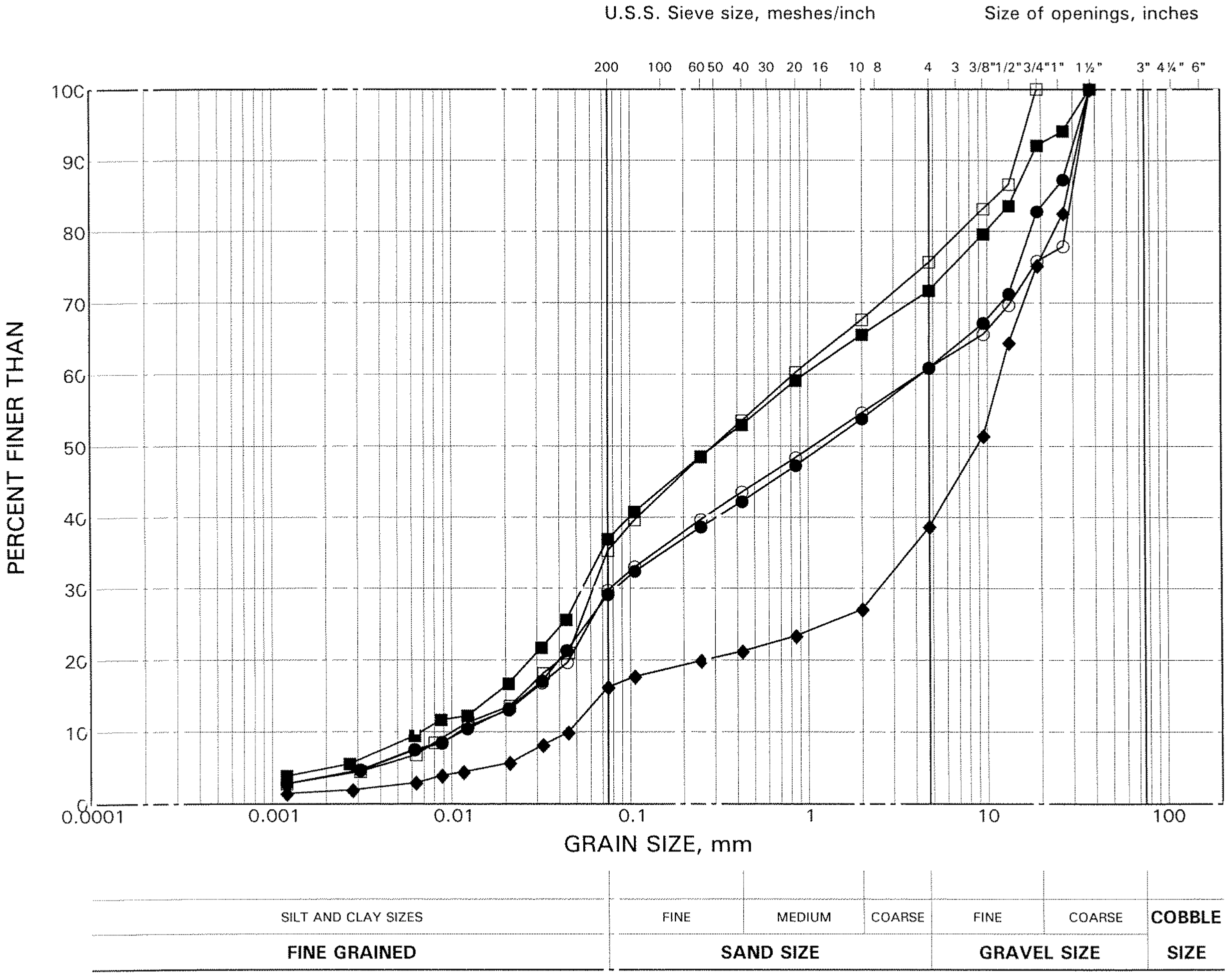
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	3	4	298.6

Project 031-111005

Golder Associates

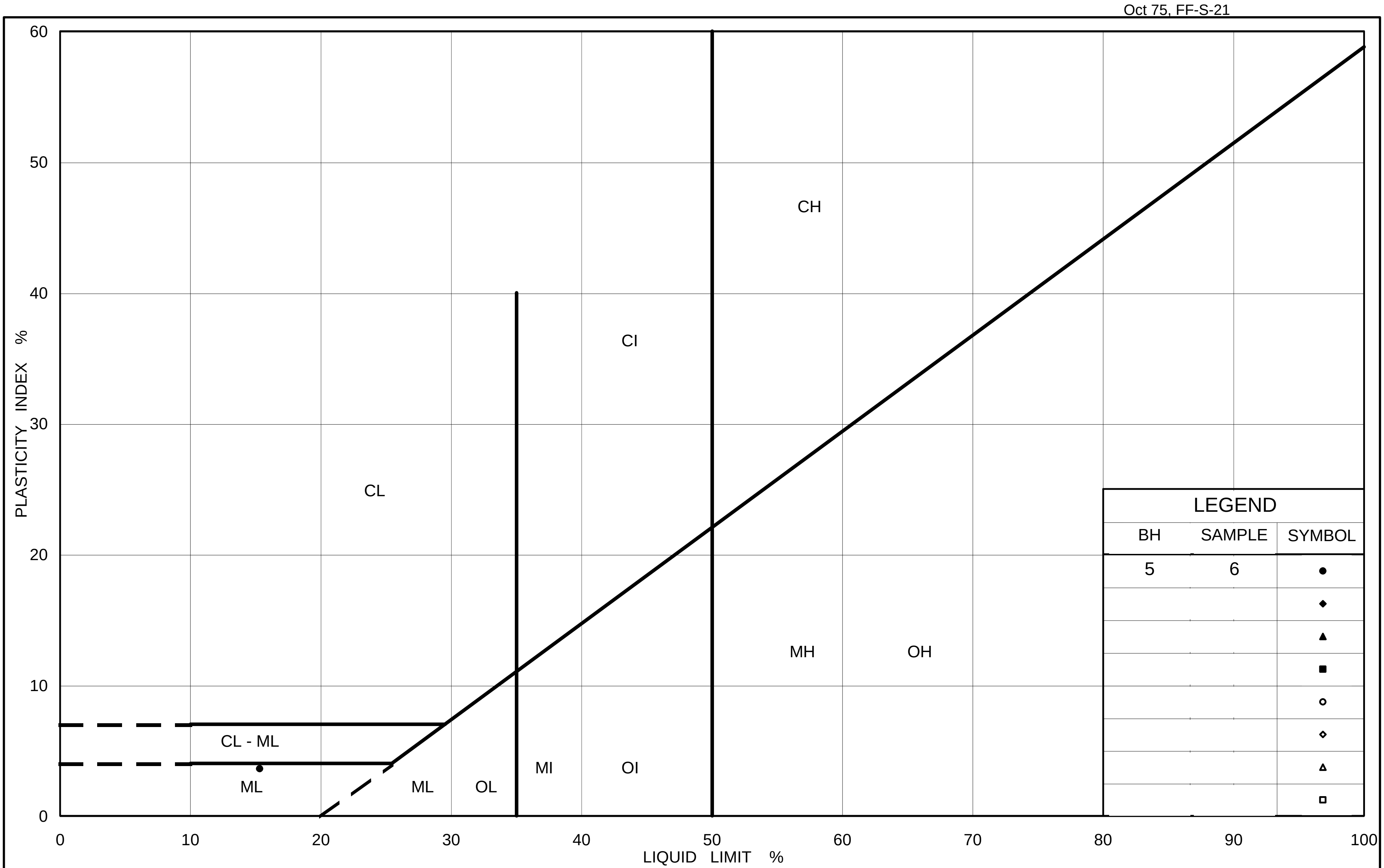
GRAIN SIZE DISTRIBUTION
Gravelly Silt and Sand to Silty Sand and Gravel (Till)

FIGURE A-2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	1	5	297.1
■	2	6	296.0
◆	4	4	297.0
○	5	5	297.0
□	6	3	298.8



Ministry of Transportation

Ontario

PLASTICITY CHART
Gravelly Silt and Sand (Till)

FIG No. A-3

Project No. 031-111005

APPENDIX B

RECORD OF BOREHOLES FROM GEOCRES 40P8-29 (1962)

BOREHOLES 1 TO 4

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 1.

JOB 62-F-107 LOCATION Sta. 34+75, 14' Lt. ORIGIN...

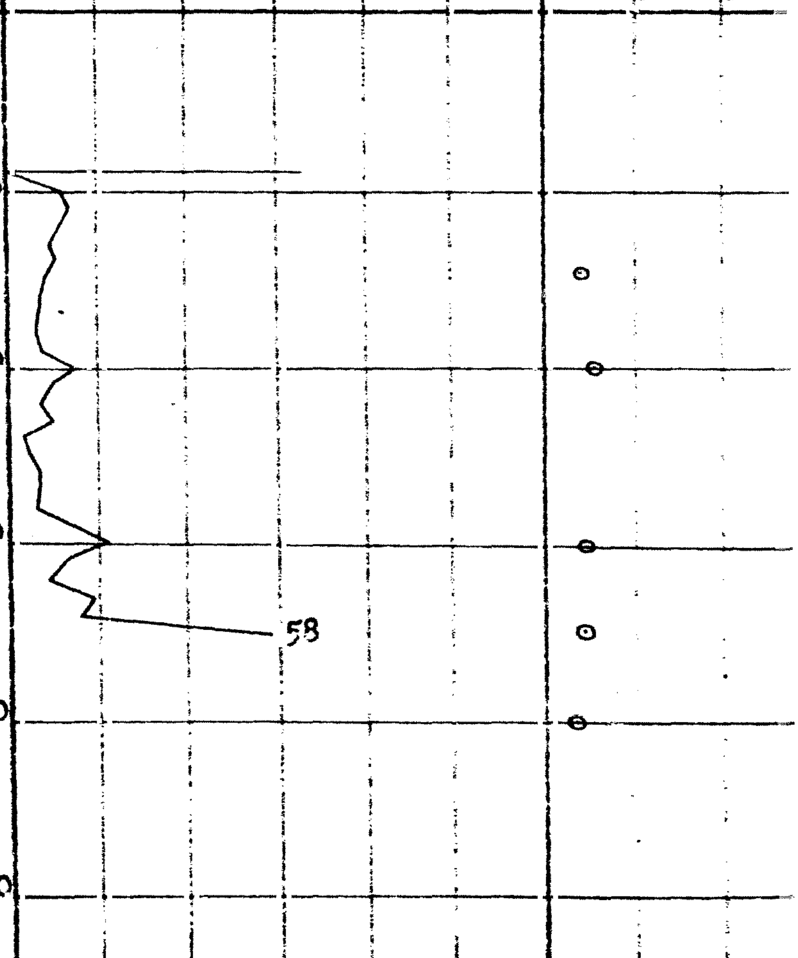
W.P 75-61 BORING DATE Sept. 10, 1962. COMPIL...

DATUM 1011.2 BOREHOLE TYPE Wash Boring - BX Casing CHECKE...

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE
30?.?? 1011.2	Ground Elevation					
00.0	Sand, Gravel, Boulders (Hwy. Fill - Compact)					1010.0
			1	SS	11	
			2	SS	18	1000.0
			3	SS	6	
300.59			4	SS	11	990.0
986.2 25.0 7.62	Dense to Very Dense Silty Sand And Gravel With Boulders Up to 18" Ø		5	SS	78	
298.16			6	SS	41	980.0
978.2 33.0 10.06	End of Borehole					
						970.0

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT
PLASTIC LIMIT
WATER CONTENT
wp w
WATER CONTE...
20 40



FORM OB-ML-[illegible] REV 1961
61-4391

OFFICE REPORT SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2.

JOB 62-F-107 LOCATION Sta. 34+41, 40' Lt. ORIGINAT[ED]

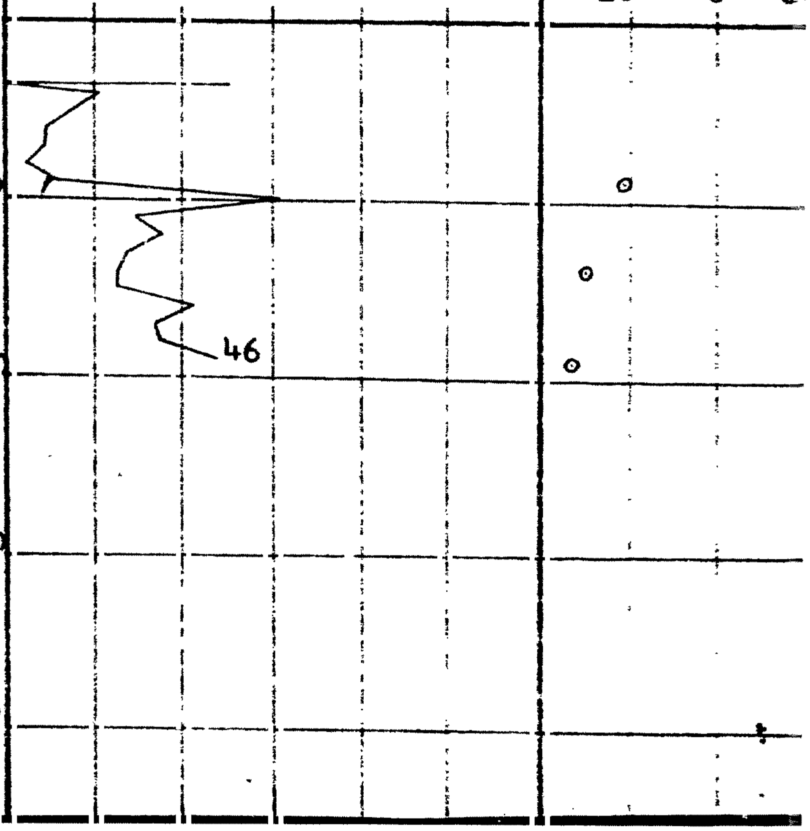
W.P. 75-61 BORING DATE Sept. 5, 1962. COMPILE[D]

DATUM 986.7 BOREHOLE TYPE Wash Boring - BX Casing CHECKED

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
300.75 986.7	Ground Elevation					
[illegible].7 1.0 0.3m	Cinders					
	Loose to Dense Silty Sand.		1	SS	8	980.0
297.7 976.7 10.0 3.0m	[illegible]se to Very Dense Silty Sand and Gravel With Boulders up to 18" Ø		2	SS	20	
			3	SS	57	970.0
294.7 966.7 20.0 6.1m	Sound Limestone Bedrock					960.0
291.9 957.7 29.0 8.8	End of Borehole					
						950.0

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT
PLASTIC LIMIT
WATER CONTENT
wp w wL
WATER CONTENT 20 40 60



FORM OB-ML-66 REV. 1961 OFFICE REPORT SOIL EXPLORATION

61-4391

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3.

JOB 62-F-107 LOCATION Sta. 33+93, 43' Lt. ORIGINAT

W.P 75-61 BORING DATE Sept. 6, 1962. COMPILE

DATUM 986.2 BOREHOLE TYPE Wash Boring - BX Casing CHECKED

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F.	LIQUID LIMIT, PLASTIC LIMIT, WATER CONTENT; WATER CONTENT 20 40 6
300.6 986.2	Ground Elevation							
985.2 1.0 0.3	Cinders							
	Loose to Dense Silty Sand		1	SS	11			o
						980.0		
297.5 976.2 10.0 3.0			2	SS	23			o
	Dense to Very Dense Silty Sand and Gravel With Boulders up to 18" Ø		3	SS	21			o
			4	SS	39	970.0		o
293.9 964.2			5	SS	45			o
22.0 6.7	End of Borehole Prob. B/R ?					960.0		

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 4.

JOB 62-F-107 LOCATION Sta. 33+61, 13' Lt. ORIGIN
W.P. 75-61 BORING DATE Sept. 12, 1962. COMPIL
DATUM 1011.7 BOREHOLE TYPE Wash Boring - BX Casing CHECKE

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
308.37 ..7 0.00	Ground Elevation				
	Sand, Gravel, Boulders. (Hwy. Fill - Compact)				1010.0
		1	SS	18	
					1000.0
		2	SS	15	
		3	SS	19	990.0
300.14 984.7 7.0 8.23		4	SS	72	
	Dense to Very Dense Silty Sand and Gravel With Boulders up to 18" Ø	5	SS	11	980.0
		6	SS	58	970.0
294.95 967.7 44.0 13.41	Sound Limestone Bedrock				
293.74 963.7 48.0 14.63	End of Borehole				960.0

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.

LIQUID LIMIT
PLASTIC LIMIT
WATER CONTENT
wp w
WATER CONTENT 20 40 6