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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
REGIONAL ROAD 20 UNDERPASS STRUCTURE
G.W.P. 275-99-00, HIGHWAY 406 TWINNING
FROM 2.2 KM NORTH OF REGIONAL ROAD 20 TO
0.2 KM NORTH OF PORT ROBINSON ROAD
CITY OF THOROLD, CENTRAL REGION**

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G.W.P. 275-99-00, HIGHWAY 406 TWINNING
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Record of Borehole Sheets (Boreholes 20-1 to 20-3, and HF-12)

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Appendix A Relevant Record of Borehole Sheets (MTO GEOCREs No. 30M3-175, dated August 1980)

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 provide foundation engineering services associated with the twinning of Highway 406 in the City of Thorold, Regional Municipality of Niagara. The project includes a new bridge structure and interchange at Regional Road 20, a pedestrian tunnel at the abandoned CN Rail, culvert extensions and high fill areas. The limits of the project extend from 0.2 km north of Port Robinson Road northerly to 2.2 km north of Regional Road 20.

This report addresses the proposed bridge structure as part of the Regional Road 20 interchange at Highway 406 that will replace the current at-grade intersection. A foundation investigation was carried out to determine the subsurface conditions at the proposed bridge location by drilling a limited number of boreholes, and carrying out in-situ testing and laboratory testing on selected samples.

The terms of reference for the scope of work are outlined in our Proposal No. P11-1346, dated October 2001. After review of the existing borehole data and proposed works, the field program was revised and discussed with the MTO Foundations group in a telephone conversation conducted on October 24, 2002. The work was carried out in accordance with Golder Associates' Quality Control Plan for Foundation Design Services, dated August 2002. A General Arrangement plan for the bridge structure was provided to Golder by Morrison Hershfield in December 2002.

The subsurface information found in the following report prepared by the MTO was utilized in the preparation of this report to supplement the subsurface data obtained during the current investigation:

- Foundation Design Report titled "Hwy 20 Overpass at Hwy 406", W.P. 88-63-00, Site 34-297, District 4, Hamilton, dated August 1980. Geocres No. 30M3-175

2.0 SITE DESCRIPTION

The site is located in the City of Thorold, Regional Municipality of Niagara, with the proposed bridge structure located about 50 m to the south of the existing at-grade intersection of Highway 406 and Regional Road 20. In this area, Highway 406 and Regional Road 20 are presently two lane, undivided highways with posted speed limits of 80 km/hr and 60 km/hr, respectively. The intersection of these highways is controlled with overhead lights.

The topography of the general area is characterized by low relief. In addition to the many wet sloughs that cover the plain as a result of poor drainage, several shallow watercourses traverse the area with direct surface drainage into several parallel streams that include Twenty Mile Creek, Forty Mile Creek and the Welland River. These watercourses are carried beneath the existing Highway 406 within the project limits with rigid frame, open footing concrete culverts that range in size from 1.2 m by 1.2 m to 2.5 m by 1.5 m. The existing Highway 406 is carried over the abandoned section of the C.N. Rail line via a three-span pre-cast concrete girder bridge that was constructed under Contract 69-137. The existing embankment is up to about 9 m high and, based on the contours shown on the topographic map, the side slopes are inclined at about 2 horizontal to 1 vertical (2H:1V).

The land within the project limits is mainly used for agricultural purposes. The perimeter of the open fields typically hold stands of mature trees. Long grass and occasional bushes cover the highway right-of-ways within the project limits.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out between October 25 and October 30, 2002. At that time, a total of three (3) boreholes, numbered Boreholes 20-1 to 20-3, were advanced at the locations of the proposed abutments and piers. In addition, Boreholes HF-12 was put down in the area of the east approach to the bridge structure. Boreholes 20-1 and 20-2 were extended into the bedrock (total depth of about 27 m and 26 m, respectively) and the remaining boreholes were extended to depths of about 5 m to 7 m. The information obtained in these boreholes was supplemented with the information obtained during the previous MTO investigation at the site. It should be noted that the numbers for the previous boreholes have been modified to reflect the year (i.e. 1980) in which they were drilled (e.g. Borehole 1 from the previous investigation designated as Borehole 80-1 herein). In addition, the boreholes from the previous investigation where only dynamic cones were carried out are referred to herein as probeholes. The previous information includes three (3) boreholes, labelled Boreholes 80-1 to 80-3, and six (6) probeholes, labelled ProbeHole 80-4 to 80-9.

The current boreholes were advanced with a track-mounted CME-75 drill rig equipped with an automatic hammer using 114 mm diameter solid stem augers supplied and operated by GeoEnvironmental Ltd. of Milton, Ontario. In the boreholes, overburden samples were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. Bedrock was cored in NQ size in Boreholes 20-1 and 20-2. The groundwater conditions in the open boreholes were observed throughout the drilling operations, and piezometers were installed in Boreholes 20-1 and 20-2 to permit monitoring of the groundwater levels at these locations. The piezometers consist of 25 mm outside diameter pipes with a 0.3 m long slotted tip that are sealed at selected depths within the boreholes. The boreholes were backfilled to ground surface with bentonite mixed with soil cuttings.

The field work was supervised on a full-time basis by a member of our engineering staff who cleared the area of buried utilities, located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples and bedrock core were identified in the field, placed in labelled containers (soil samples) and boxes (bedrock core), and transported to our laboratory in Mississauga for further examination and testing. Index and classification tests consisting of grain size analyses, Atterberg limits tests and water content determinations were carried out on selected soil samples. Point load tests were also carried out on selected sections of the recovered core.

The borehole locations were established relative to the proposed limits of the foundation units and Regional Road 20 centreline stakes in the field. The as-drilled borehole locations and elevations were surveyed by Callon Dietz Inc. of London, Ontario. It is understood that the northing and easting coordinates are referenced to the MTM coordinate system and that the

elevations are referenced to Geodetic Datum. The borehole locations for the current and previous investigations are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published literature, the site is located within the physiographic region known as the Haldimand Clay Plain (“The Physiography of Southern Ontario”, 3rd Edition, Chapman and Putnam, 1984). This region was submerged by glacial Lake Warren and as such much of the subsoil is comprised of stratified lacustrine silts and clays. In some areas the stratified clay overlies clayey till while in other areas the subsoil is represented by an interlayered / intermixed deposit of lacustrine silt / clay and till. The overburden is generally less than about 20 m thick, with a trend of increasing thickness towards Lake Erie. The underlying bedrock consists of a succession of Paleozoic beds dipping slightly southward under Lake Erie. Dolostone is the predominate type of rock within the plain with softer, shaley rock found in the southwest area of the plain.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil and bedrock samples, are given on the attached Record of Borehole and Corehole sheets and on Figures 1 to 5 following the text of this report. The stratigraphic boundaries shown on the borehole and corehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil and bedrock conditions will vary between and beyond the borehole / corehole locations.

In summary, the subsoils at the site consist of a surficial layer of topsoil and / or granular fill underlain by an upper deposit of silty clay, which in turn is underlain by clayey silt with occasional silty clay and silt seams / layers. A heterogeneous mixture of silty clay, sand and gravel was encountered beneath the clayey silt and this deposit overlies Dolomite bedrock. The aquifer in the bedrock controls the primary groundwater level, which was measured in the piezometers at about 8 m to 9 m depth about two weeks after their installation.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. The locations and elevations of the current and previous boreholes are shown on the attached Drawing 1. The Record of Borehole Sheets for the previous investigation at the site are included in Appendix A.

4.2.1 Topsoil

A 0.1 m to 0.2 m thick surficial layer of topsoil was encountered in all of the boreholes, except Borehole 20-2.

4.2.2 Fill Materials

Fill materials were encountered beneath the topsoil in Borehole 20-1 and surficially in Boreholes 20-2, 80-1 and 80-2. The fill consists of silty clay, trace sand, gravel, and topsoil at the location of Borehole 20-1 where the layer is 0.6 m thick. Standard Penetration Testing (SPT) carried out within this fill layer measured an 'N' value of 23 blows per 0.3 m of penetration, which indicates a very stiff consistency. A 0.3 m thick layer of rockfill was encountered at the location of Borehole 20-2, which was put down within a shallow ditch adjacent to the roadway; the rockfill was likely placed for erosion protection measures at this location. A 0.7 m and 1 m thick layer of sand and gravel fill was encountered in Boreholes 80-1 and 80-2, respectively. This material was placed during the construction of the old Regional Road 20 that existed before the current Regional Road 20 was built.

4.2.3 Upper Silty Clay

A deposit of reddish brown / grey silty clay, trace sand and gravel was encountered below the topsoil and fill materials in all the boreholes. The deposit is typically weathered throughout its 3 m to 5 m thickness where oxidation staining and fissuring were noted in the samples. A grain size distribution curve of a selected sample of the silty clay is shown on Figure 1.

Standard Penetration Testing (SPT) measured 'N' values ranging from 4 blows to greater than 50 blows per 0.3 m of penetration, which indicates a firm to hard consistency. In general, the upper silty clay is hard as a result of weathering that has formed a crust. Field vane testing was attempted within the deposit; however, it was found that the vane could not be turned. This indicates that the undrained shear strength exceeds 96 kPa (i.e. the upper limit of shear strength measurement of the vane).

Atterberg Limits testing conducted on a selected sample obtained from this stratum show a liquid limit (w_L) of about 40 percent and a plasticity index (I_p) of about 21 percent. The results of the Atterberg limits testing carried out on samples of this deposit as part of the previous investigation show liquid limits (w_L) ranging from about 32 percent to 47 percent and plasticity indices (I_p) ranging from about 17 percent to 19 percent. The results of the Atterberg Limits testing classify the soil in this stratum as an inorganic silty clay of low to intermediate plasticity. The results of the current Atterberg Limits testing are shown plotted on the plasticity chart on Figure 2. The natural water content measured on selected samples from this stratum range from about 14 percent to 23 percent, with an average of about 19 percent. In general, the water contents were found to be at or slightly above the measured plastic limits, corresponding to a liquidity index of less than 1.

4.2.4 Lower Clayey Silt

The predominant deposit at the site was encountered beneath the silty clay stratum and consists of brown to reddish grey clayey silt with occasional silty clay and silt seams / layers and containing trace of sand and gravel. A grain size distribution curve of a selected sample of the clayey silt is shown on Figure 3.

Standard Penetration Testing (SPT) indicates a wide range of scatter within the clayey silt with 'N' values measured between 13 blows to greater than 50 blows per 0.3 m of penetration, indicating a stiff to hard consistency. In general, the lower clayey silt is very stiff. Field vane testing was attempted within the deposit; however, it was found that the vane could not be turned. This indicates that the undrained shear strength exceeds 96 kPa.

Atterberg Limits testing conducted on selected samples obtained from this stratum indicate liquid limits (w_L) of between about 14 percent and 16 percent and plasticity indices (I_p) of between about 5 percent and 11 percent. A non-plastic result was measured on a sample of the silt layer encountered at about 16.3 m to 17.5 m depth in Borehole 20-2 (Sample 13). The results of the Atterberg limits testing carried out on samples of this stratum obtained for the previous investigation show liquid limits (w_L) ranging from about 18 percent to 22 percent and plasticity indices (I_p) ranging from about 4 percent to 7 percent. The results of the Atterberg Limits testing classify this stratum as an inorganic clayey silt of low plasticity. The results of the current Atterberg Limits testing are shown on the plasticity chart on Figure 4. The natural water content measured on selected samples from this stratum typically range from about 12 percent to 23 percent, with an average of about 15 percent. In general, the water contents were found to be at or slightly above the measured plastic limits, corresponding to a liquidity index of less than 1.

The clayey silt deposit was fully penetrated in Boreholes 20-1, 20-2 and 80-1 to 80-3 where it was found to be about 14.6 to 17.2 m thick, corresponding to an Elevation of about 162.5 m at the base of the deposit.

4.2.5 Heterogeneous Mixture of Silty Clay, Sand and Gravel

Beneath the clayey silt stratum exists a heterogeneous mixture of reddish grey silty clay, sand and gravel that ranges in thickness from 1 m to 2.8 m at the borehole locations. Pieces of dolomite gravel were also noted within this deposit. A grain size distribution curve of a selected sample of this stratum is shown on Figure 5. This sample shows a predominance (about 80 percent) of sand and gravel. Standard Penetration Testing (SPT) measured 'N' values that exceed 100 blows per 0.3 m of penetration, indicating a hard consistency / very dense state of packing. The natural water content measured on selected samples from this stratum typically range from about 8 percent to 20 percent, with an average of about 11 percent.

Although not encountered in the boreholes, cobbles and/or boulders should be anticipated within this deposit.

4.2.6 Bedrock

Bedrock was encountered at between Elevations 160.5 m and 162.2 m in Boreholes 20-1, 20-2, 80-2, and 80-3 (about 21.3 to 23.3 m below the ground surface). Bedrock was inferred at 21.6 m depth (Elevation 161.7 m) in Borehole 80-1 based on auger refusal. Boreholes 20-1 and 20-2 of the current investigation were advanced about 3.8 m and 3.2 m, respectively, into the bedrock by coring in NQ size. Boreholes 80-2 and to 80-3 from the previous investigation were advanced about 1.4 m and 1.9 m, respectively, into bedrock. The rock core samples consist of slightly weathered to fresh, grey, medium to coarse-grained, strong, mainly massive with some thin bedding Dolostone. Very few fractures exist within the recovered core. Where fractures were encountered, the fracture index is between 1 and 3 fractures per 0.3 m. The Rock Quality Designation (RQD) measured on the core samples range from about 85 percent to 100 percent, which indicates that the rock mass is of good to excellent quality based on the guidelines provided in the Canadian Foundation Engineering Manual (CFEM, 3rd Edition, 1992). In general, the rock mass quality is excellent. Strength testing carried out on four samples of the recovered core gave diametrical point load indices of between about 3.5 MPa and 6.2 MPa, which corresponds to Unconfined Compressive Strengths (UCS) of between about 80.5 MPa and 142.5 MPa.

4.2.7 Groundwater Conditions

Water was introduced into Boreholes 20-1 and 20-2 for coring the bedrock and therefore water level readings were not obtained upon completion of drilling of these boreholes. Borehole 20-3 was open and dry upon completion of drilling. The base of Borehole HF-12 was wet upon completion of drilling. Wet conditions were also encountered in Borehole 20-1 at about 4.9 m depth (Elevation 178.9 m) and in Borehole 20-2 at about 6.7 depth (Elevation 176.4) based on the samples obtained. A piezometer was sealed into the bedrock and overlying heterogeneous mixture of silty clay, sand and gravel in each of Boreholes 20-1 and 20-2. The water level measured in the piezometer installed in Borehole 20-1 was at 9.1 m depth (Elevation 174.4 m) about 2 weeks and again at 3 months after installation. The water level measured in the piezometer installed in Borehole 20-2 was at about 7.8 m depth (Elevation 175.3 m) and 6.5 m depth (Elevation 176.6 m) about 2 weeks and 3 months, respectively, after installation. These levels indicate that the groundwater table slopes down towards the west.

Piezometers installed at other locations within the project limits indicate that a perched water condition exists within the silty clay and upper portion of the clayey silt. The perched water level follows the ground surface within the project limits and is generally within about 2.5 m of the ground surface. This perched condition will vary with the amount of precipitation.

Piezometers were not installed in the previous Boreholes 80-1 to 80-3. However, these boreholes were left open for a period of seven days at which time the water levels in the boreholes were measured. Water was found at about 1.8 m depth (about Elevation 180.5 m) in these boreholes. The groundwater information in the Preliminary Design and Environmental Assessment Report (dated September 2001) for this project indicates that there are no significant shallow aquifers within the study area and that the primary aquifer(s) is found within the bedrock. The groundwater level was generally found below 8 m depth based on the well records in the area as part of the previous study.

It should also be noted that the groundwater levels in the area are subject to seasonal variations and are expected to be higher during wet periods of the year.

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PART B

**FOUNDATION DESIGN REPORT
REGIONAL ROAD 20 UNDERPASS STRUCTURE
G.W.P. 275-99-00, HIGHWAY 406 TWINNING
FROM 2.2 KM NORTH OF REGIONAL ROAD 20 TO
0.2 KM NORTH OF PORT ROBINSON ROAD
CITY OF THOROLD, CENTRAL REGION**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed Highway 406 Underpass at the realigned Regional Road 20. The recommendations are based on interpretation of the factual data obtained from a limited number of boreholes advanced during the current and previous subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives for design of the proposed underpass and staging of this construction. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project.

The limits of this project extend from 0.2 km north of Port Robinson Road northerly to 2.2 km north of Regional Road 20. Within these limits, it is understood that Highway 406 will be twinned with the existing two lanes of the highway converted to the northbound lanes and the construction of two new southbound lanes. It is proposed to realign Regional Road 20 to the south of its existing location in the area of Highway 406 and replace the existing at-grade intersection with a two-span, 80 m long underpass structure. Regional Road 20 will also have an ultimate configuration of four lanes. According to the Preliminary General Arrangement drawing provided by Morrison Hershfield, dated November 2002, the approach embankments will be up to 8.5 m high with RSS walls forming the wing walls of the bridge. The bridge deck will dip slightly from the west to the east (about 1 m over its 80 m length). The bridge will carry the proposed four lanes of Regional Road 20 and lanes for the W-N and E-S ramps over Highway 406. The General Arrangement Drawing indicates that the bridge deck will be about 40 m wide in order to accommodate these lanes.

The existing subsurface information, which includes Borehole 80-1 and Probeholes 80-4 to 80-9 was used to supplement the information from Borehole 20-1 at the location of the proposed west abutment. The existing information (Borehole 80-3) was also used to assess the foundation conditions at the proposed east abutment. A second borehole was not put down at this abutment alignment due to the presence of buried utilities in the area of the proposed footing. Boreholes 20-2 and 20-3 were extended to address the centre pier location. Borehole 20-2 was extended into the bedrock and Borehole 20-3 was extended below the "crust" to assess the thickness at this location. These boreholes were put down as close as possible to the proposed pier alignment outside the limits of the existing ramp from Regional Road 20 eastbound to Highway 406 southbound.

5.2 Bridge and Retaining Wall Foundation Options

At the site of the proposed structure, the natural ground surface is relatively flat and is typically found at about Elevation 183.5 m. The existing Highway 406 road structure consists of a low embankment that is about 1 m above the surrounding ground.

The subsoils at the site consist of a surficial layer of topsoil and granular fill / rockfill underlain by an upper stratum of generally hard silty clay of low to medium plasticity that is about 3 m to 5 m thick at the borehole locations. The silty clay is underlain by the predominant deposit at the site, which is comprised of generally very stiff clayey silt of low plasticity that extends to about Elevation 162.5 m. Silty clay and silt seams / layers are randomly found throughout the deposit. Beneath the clayey silt exists a heterogeneous deposit of hard / very dense silty clay, sand and gravel that is up to about 2.8 m thick at the borehole locations. Dolostone bedrock is found at about Elevations 160.5 m to 162.2 m at the borehole locations. The groundwater level was measured in the piezometer in Borehole 20-1 at 9.1 m depth (Elevation 174.4 m) about 2 weeks and 3 months after the installation of the piezometer. The ground water level was measured in the piezometer in Borehole 20-2 at about 7.8 m depth (Elevation 175.3 m) and 6.5 m depth (Elevation 176.6 m) about 2 weeks and 3 months, respectively, after installation of the piezometers. Both piezometers were sealed within the bedrock and more permeable stratum that immediately overlies the bedrock. Based on the observations made in the previous boreholes (which were left open and measured after 7 days) and in the current boreholes (wet conditions and water at the base of Borehole HF-12, and other piezometers within the project limits), a perched water condition exists which is governed by surface water infiltration through the upper fissured silty clay deposit.

The native soils at the site are suitable for support of the proposed abutments and associated retaining wall with shallow foundations. However, the randomly distributed “weaker” zones within the soil mass (as identified by lower ‘N’ values) and areas within the soil that have medium plasticity will restrict the bearing capacity of the soil. Additionally, the compressive nature of the clayey soil will result in more consolidation settlement at the abutments (due to the embankment loading) and less consolidation settlement at the pier location, resulting in differential settlement along the structure. As an alternative, deep foundations, such as steel H-piles driven to bedrock, are considered feasible to support the proposed two-span structure.

Preloading or the use of lightweight for the approach embankment areas is a feasible option for reducing consolidation settlement at this site. Due to the nature of the subsurface conditions, wick drains are not considered to be a viable option.

Recommendations for both shallow and deep foundations for the bridge abutments and pier are presented in the following sections.

5.3 Spread Footings

The bridge abutments and pier may be supported on spread footings placed on the native soils below any topsoil and fill. Wing walls may be supported on the native soils or within the embankment fill. The GA drawings provided indicate that the wing walls will be stepped up along their length, founded within the embankment fill.

5.3.1 Axial Geotechnical Resistance

It is assumed that the top of the spread footings for the abutments and pier would be located about 1 m below the existing ground surface and that the footings would be about 1.5 m thick. Therefore, the base of the footings would be located within the upper, more competent (due to weathering) silty clay at least 0.5 m, and up to about 2 m, above the underlying less competent clayey silt (elevation of the base of the proposed footings at about 181 m). Spread footings placed within the undisturbed silty clay deposit, at Elevation 181 m, may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 350 kPa. The settlement of the footings will be dependent on the footing size, configuration, applied loads and embankment loading. The geotechnical resistance at Serviceability Limit States (SLS) may be taken as 175 kPa. This SLS value applies to the centre pier where consolidation from the embankment loading will not occur. This value would also apply to the abutments in the case that the embankment is constructed first and left to consolidate the underlying clayey soil for a period of at least 3 months (see Section 5.8).

These geotechnical resistances assume a footing width of 5 m and a footing length of about 40 m. The geotechnical resistances should be reviewed if there are significant changes in the foundation geometry.

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 the *Canadian Highway Bridge Design Code (CHBDC)*.

5.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$ or $\tan \phi'$, may be taken as 0.45 for cast-in-place concrete footings constructed on the undisturbed, generally hard silty clay. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.3.3 Frost Protection

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

5.3.4 Construction Considerations

Perched water may exist within the silty zones in the silty clay deposit and therefore some water inflow during and upon completion of the excavation for the footings may occur. It is expected that the quantity of flow will 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 sensitive to disturbance and softening due to water seepage or ponding. Placement of a lean concrete coat will be required at the base of excavation for the footing area. Exposure without protection of the lean concrete coat will allow water to soften the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the lean concrete coat. The lean concrete should be placed within four hours after footing inspection.

5.4 Driven Steel H-Pile Foundations

The use of deep foundations at this site is considered to be a practical option and consideration could be given to the use of steel 310 x 110 H-piles driven to refusal on the competent bedrock. The following table provides the design tip elevations at the proposed foundation units.

Location	Design Tip Elevation (m)
East Abutment	162.0 ±
Centre Pier	160.5 ±
West Abutment	160.5 ±

The presence of a 1 m to 2.8 m thick heterogeneous mixture of hard / very dense silty clay, sand and gravel with pieces of dolomite (characterized by SPT 'N' values that exceed 100 blows / 0.3 m) will result in heavy driving to get the H-piles seated on the bedrock. Stiffening of the pile toe with flange plates as well as stiffening of the top will be required for protection during driving. The following note should be added to the contract drawings: "Piles to be driven to bedrock".

5.4.1 Axial Geotechnical Resistance

The factored axial resistance at ULS for steel HP 310 x 110 piles driven to refusal on the dolomite bedrock may be taken as 2,000 kN. There will be negative skin friction induced on the abutment piles from consolidation of the clayey subsoils under the embankment loading. The consequent unfactored downdrag load on the piles at the abutments is estimated at 1,000 kN. Applying a resistance factor of 0.4 (as per CHBDC Table 6.6.2.1) on the unfactored axial resistance for steel HP310x110 piles of 5,000 kN, the net factored axial resistance at ULS is 1,600 kN [= (5,000 kN-1,000 kN) x 0.4].

Refusal to further pile penetration may be reached within the hard / very dense heterogeneous deposit that overlies the bedrock. For this condition of piles “hanging up” within this stratum, a lower design capacity would apply and a net factored axial resistance at ULS of 1,400 kN should be assumed. Serviceability Limit States (SLS) do not apply to piles driven to refusal at this site.

The effect of the downdrag can be reduced or even eliminated if the embankments are constructed and the underlying clayey subsoils are allowed to consolidate prior to installation of the piles at the abutments.

5.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

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

$$k_h = \frac{k_{s1}}{5B} \quad \text{where } B \text{ is the pile diameter (m) and}$$

k_{s1} is the coefficient of horizontal subgrade reaction, as given below.

The following ranges for the value of k_{s1} may be assumed in the structural analysis:

<i>Soil Unit</i>	<i>k_{st}</i>
Generally hard silty clay above about Elevation 179.5 m	50 to 100 MPa/m
Very stiff clayey silt below about Elevation 179.5 m and above about Elevation 162.5 m	30 to 60 MPa/m
Hard / very dense silty clay, sand and gravel below about Elevation 162.5 m	150 MPa/m

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than about 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:

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

5.4.3 Frost Protection

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

5.4.4 Construction Considerations

The construction considerations outlined in Section 5.3.4 for spread footings should be followed for construction of the pile caps.

5.5 Retained Soil System (RSS) Walls

Settlement of the embankment and walls at this site will occur due to consolidation of the subsoils. Use of an RSS wall is considered appropriate for the proposed wing walls / retaining walls, provided that sufficient preloading is carried out to reduce the settlement of the embankments to within tolerable limits.

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.

Based on the configuration shown on the GA drawings, the walls will be stepped up within the embankment fill and will be about 2 m to 4 m high. A typical RSS wall is founded at least 0.3 m below the existing ground surface in front of the wall. The walls will be founded either on the native silty clay or within the embankment fill. For the reinforced earth mass founded on the very stiff to hard silty clay or within granular embankment fill material, the factored geotechnical resistance at ULS will depend on the width of the reinforced soil mass and the following values may be used for design:

- 60 kPa for a 2 m high wall; and
- 120 kPa for a 4 m high wall.

These values assume 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. As discussed under Section 5.8, the construction of the embankment will induce consolidation of the underlying clayey soils. The geotechnical resistance at SLS, for 25 mm of settlement, will be governed by the embankment settlement and consolidation settlement of the underlying founding soils. It is recommended that the granular fill be used for embankment construction where this fill will form the founding stratum. Provided that the embankment is constructed first and left to consolidate the underlying subsoil for a period of at least 3 months prior to construction of the RSS wall (see Section 5.8), the geotechnical resistance at SLS will be greater than the ULS values given above and these values may be used for design. In order to carry this out, the fill material in the area of the RSS wall would have to be removed after the consolidation period and the wall subsequently constructed with granular fill. If the wall is constructed immediately as part of the embankment, the wall would be subjected to the embankment settlements outlined in Section 5.8, and would likely experience joint separation and / or cracking.

The resistance to lateral forces / sliding resistance between the compacted Granular “A” and the clayey subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$ or $\tan \phi'$, between the compacted Granular “A” of the RSS wall and the very stiff to hard silty clay may be taken as 0.45. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

The internal stability of the mechanically-reinforced soil walls should be checked by the RSS supplier / designer. The Factor of Safety related to global stability for properly designed and constructed RSS walls at this site will be greater than 1.3.

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:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve 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. 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 assumed based on the use of Granular 'B' Type II as required for support of RSS walls:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	0.31
Active, K_a	0.47
At rest, K_o	

- 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'
Soil unit weight:	22 kN/m ³	Type II 21 kN/m ³
Coefficients of 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.

It should be noted that the above 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.

5.7 Excavations and Temporary Cut Slopes

Excavations for construction of the abutment, pier and wing wall / retaining wall footings or pile caps will extend through the fills and into the upper silty clay to an estimated depth of 3.5 m below the existing ground surface. The excavation will extend below the depth where standing water was observed in the open boreholes put down as part of the previous investigation that reflects a perched water condition. The excavation bases, however, will be above the groundwater table measured in the piezometers sealed in the bedrock and the more permeable overburden immediately above the bedrock, which reflects the primary groundwater table.

5.7.1 Open Cut Slopes

Temporary open cut slopes should be maintained no steeper than 3 horizontal to 1 vertical (3H:1V) within the fill materials and 1H:1V within the native materials for the estimated depth of cut equal to 3.5 m below the ground surface. Where space restrictions dictate, the excavation could also be carried out within a fully braced excavation. The excavation for spread footing or pile cap construction adjacent to the existing Highway 406 may have to be made with vertical supported sides to minimize disruption to road traffic and provide lateral support to the adjacent roadway (see Section 5.7.2).

Water seepage into the excavations through the fill and upper silty clay deposit is expected to be minor, except during periods of sustained precipitation. Pumping from well-filtered sumps located at the base of the excavation should provide adequate groundwater control during foundation excavations. The consideration with respect to protection of the founding soils, however, as given in Section 5.3.4 must be recognized. Sumps should be maintained outside the

actual footing limits. Surface water run-off should be directed away from the excavations at all times. The appropriate NSSP should be included in the contract documents.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health & Safety Act. The native soils at this site would be classified as Type I soil. The fill deposits would be classified as Type III soil.

5.7.2 Temporary Excavation Support

Where space restrictions (e.g. near the existing Highway 406) or construction staging requirements preclude the use of temporary open-cuts, a temporary excavation support system will be required to construct the footings or pile caps. The temporary excavation support system should be in accordance with Special Provision 539S01 and be designed to Performance Level 2 as defined by this Provision.

Roadway protection should be as per Special Provision 539S01.

5.8 Approach Embankment Design

The construction of the underpass structure and approach embankments will require placement of up to 8.5 m of fill material. Based on the borehole results, the embankment subgrade soils will consist of very stiff to hard silty clay underlain by very stiff clayey silt. All topsoil / organic matter, fill, and softened / loosened soils should be stripped from beneath the proposed approach embankment envelopes, and all subgrade soils should be proof-rolled prior to fill placement. The topsoil ranges in thickness from 0.1 m to 0.2 m at the borehole locations.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting specifications OPSS 212 or Selected Subgrade Material meeting specifications with OPSS 1010, depending on material availability. In the areas where RSS walls are to be founded within the embankment fill, granular fill should be used. All 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 geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

A stability analysis was carried out to assess the stability of the proposed embankment slopes using the commercially available program SLOPE/W (Version 5.13) produced by GEO-SLOPE International Ltd. employing the General Limit Equilibrium method of analysis. The factor of

safety of numerous potential failure surfaces were computed in order to assess minimum factors of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A factor of safety of at least 1.3 is considered acceptable for the overall long-term stability of the embankments.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, embankments up to about 8.5 m in height with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have an adequate Factor of Safety (F.S.) against deep-seated slope instability (i.e. $F.S. \geq 1.3$). The result of the stability analysis carried out for the proposed approach embankments is shown on Figures 6 and 6A. Both effective stress (drained) and total stress (undrained) conditions were analysed. The analysis considered the case where only the embankment is present (expected to be less than 8 m in height) and the case where the RSS wall is present as part of the embankment structure (expected to be up to about 8.5 m in height). If there are areas with embankment side slopes greater than 8 m in height, a 2 m wide mid-height berm should be provided. To reduce surface water erosion, placement of topsoil and seeding or pegged sod is recommended as per OPSS 572.

Long-term settlement of the proposed embankment will occur due to the consolidation of the underlying clayey soils. The amount of settlement will vary according to the thickness of fill placed to construct the new embankment. Settlement analyses were carried out using Compression and Recompression Index (C_c and C_r) profiles obtained from empirical correlations with the laboratory data (Atterberg Limits). The deposit is overconsolidated based on the results of the consolidation tests (oedometer) carried out on two samples obtained at the CN Rail site within the project limits (Golder Report No. 021-1143-2, dated July 2003). It was found that the resulting effective stress after embankment loading is still less than the preconsolidation pressure (i.e. only the C_r profile applies). The maximum settlement, corresponding to the area where the highest embankment will be constructed (i.e. close to the bridge abutments) is estimated to be up to 90 mm as shown on Figure 7. It is estimated that 70 percent of the consolidation (about 65 mm of settlement) will occur within 3 months of construction and that 90 percent of the consolidation (about 80 mm of settlement) will occur within 6 months. This is based on a coefficient of consolidation, c_v , equal to 0.06 cm/s^2 , which was measured for a similar soil sample obtained at the CN Rail site for the anticipated stress conditions. The settlement related to the compression of the fill materials comprising the embankment is anticipated to be minor, and will occur during construction.

6.0 CLOSURE

This report was prepared by Dan Breeze, P.Eng., Project Engineer, under the supervision of Anne Poschmann, P.Eng., Project Manager. The overall review and quality control of this project was carried out by Fin Heffernan, P.Eng., Designated MTO Contact.

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DKB/ASP/FJH/mmh

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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

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

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.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

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, 60° 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 cm² 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.

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.

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

PROJECT 021-1143

RECORD OF BOREHOLE No 20-1

1 OF 2

METRIC

W.P. 275-99-00

LOCATION N 4769665.3; E 326326.6

ORIGINATED BY PKS

DIST 4 HWY 406

BOREHOLE TYPE Solid Stem Augers

COMPILED BY DKB

DATUM Geodetic

DATE October 25&28, 2002

CHECKED BY ASP

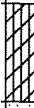


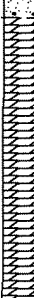
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
183.8	GROUND SURFACE							20 40 60 80 100				
0.0	Topsoil		1	SS	23		183	20 40 60 80 100				
183.1	Silty Clay, trace sand and gravel, trace topsoil (Fill)		2	SS	25		182	20 40 60 80 100				
0.7	Very stiff Brown Moist		3	SS	35		181	20 40 60 80 100				
	Silty Clay, trace sand and gravel, occasional oxidation staining, fissured		4	SS	35		180	20 40 60 80 100				
	Very stiff to hard Reddish brown/grey Moist		5	SS	65		179	20 40 60 80 100				
179.7	Clayey Silt, trace sand and gravel, occasional silty clay seams/layers		6	SS	20		178	20 40 60 80 100				
4.1	Very stiff Brown to reddish grey Moist to Wet below 4.9m depth		7	SS	29		177	20 40 60 80 100				
178.2	Clayey Silt, trace sand and gravel		8	SS	20		176	20 40 60 80 100				
5.6	Very stiff to hard Reddish grey Wet		9	SS	22		175	20 40 60 80 100				
			10	SS	22		174	20 40 60 80 100				
			11	SS	25		173	20 40 60 80 100				
			12	SS	21		172	20 40 60 80 100				
			13	SS	20		171	20 40 60 80 100				
			14	SS	50		170	20 40 60 80 100				
			15	SS	32		169	20 40 60 80 100				
							168	20 40 60 80 100				
							167	20 40 60 80 100				
							166	20 40 60 80 100				
							165	20 40 60 80 100				
							164	20 40 60 80 100				

Silt layer between 16.7m and 17.4m depth

Continued Next Page

+³, X³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

PROJECT <u>021-1143</u>		RECORD OF BOREHOLE No 20-1		2 OF 2	METRIC
W.P. <u>275-99-00</u>	LOCATION <u>N 4769665.3; E 326326.6</u>	ORIGINATED BY <u>PKS</u>			
DIST <u>4</u> HWY <u>406</u>	BOREHOLE TYPE <u>Solid Stem Augers</u>	COMPILED BY <u>DKB</u>			
DATUM <u>Geodetic</u>	DATE <u>October 25&28, 2002</u>	CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p W W _L				
	--- CONTINUED FROM PREVIOUS PAGE ---													
162.5 21.3	Clayey Silt, trace sand and gravel Very stiff to hard Reddish grey Wet		16	SS	32		163							
	Hetrogenous mixture of SILTY CLAY, SAND and GRAVEL Hard/Very Dense Reddish grey Wet		17	SS	100/0.2		162							
160.5 23.3			18	SS	100/0.03		161							
	Fresh to slightly weathered, strong, grey, medium to coarse grained DOLOSTONE, very few fractures, some vuggs, some fossils, massive to thinly bedded.						160							
							150							
							150							
156.7 27.1	For bedrock coring details refer to Record of Drillhole 20-1.						157							
	END OF HOLE													
	Notes: 1. Water level not obtained in open borehole upon completion of drilling due to use of water for coring. 2. Water level measured in piezometer at 9.1m depth (El.174.4m) on Nov.15, 2002 and on Jan.31, 2003.													

SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: Geo-Environmental Ltd.

[illegible]

PROJECT 021-1143		RECORD OF BOREHOLE No 20-2		1 OF 2	METRIC
W.P. 275-99-00		LOCATION N 4769691.8; E 326359.7		ORIGINATED BY PKS	
DIST 4 HWY 406		BOREHOLE TYPE Solid Stem Augers		COMPILED BY DKB	
DATUM Geodetic		DATE October 28,29,2002		CHECKED BY ASP	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P	W	W _L		
183.1	GROUND SURFACE						183							
0.0	Rockfill						182							
0.3	Silty Clay, trace sand and gravel, occasional oxidation staining, fissured		1	SS	29		181							
	Very stiff		2	SS	25									
	Reddish brown/grey		3	SS	20									
180.1							180							
3.0	Clayey Silt, trace sand and gravel		4	SS	26		179							
	Very stiff to hard						178							
	Brown to Grey to Reddish grey		5	SS	30		177							
	Moist to wet		6	SS	27		176							
	below 6.7m depth		7	SS	24		175							
			8	SS	28		174							
			9	SS	27		173							
			10	SS	22		172							
			11	SS	38		171							
			12	SS	100/45		170							
			13	SS	48		169							
			14	SS	31		168							
			15	SS	100/45		167							
							166							
							165							
							164							
163.3														

Silt layer from 16.3m to 17.5m depth.
Non-plastic Atterberg Limits result measured for Sample 13

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE

MISS_MTO 021-1143.GPJ ON MOT GDT 27/5/03

PROJECT <u>021-1143</u>		RECORD OF BOREHOLE No 20-2		2 OF 2	METRIC
W.P. <u>275-99-00</u>	LOCATION <u>N 4769691.8; E 326359.7</u>	ORIGINATED BY <u>PKS</u>			
DIST <u>4</u> HWY <u>406</u>	BOREHOLE TYPE <u>Solid Stem Augers</u>	COMPILED BY <u>DKB</u>			
DATUM <u>Geodetic</u>	DATE <u>October 28, 29, 2002</u>	CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								<div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div></div><div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PROJECT: 021-1143

RECORD OF DRILLHOLE: 20-2

SHEET 3 OF 3

LOCATION: N 4769691.8; E 326359.7

DRILLING DATE: October 29, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME - 75

DRILLING CONTRACTOR: Geo-Environmental Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLOUR % RETURN	FR/FX-FRACTURE F-FAULT				SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE		DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
				DEPTH (m)					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					
									RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY					
TOTAL CORE %	SOLID CORE %	TYPE AND SURFACE DESCRIPTION	K, cm/sec																	
80	90			10 ⁻⁶	10 ⁻⁵															
90	100			10 ⁻⁴	10 ⁻³															
		GROUND SURFACE																		
23		Fresh to slightly weathered, strong, grey, medium to coarse grained DOLOSTONE, very few fractures, some vuggs, some fossils, mainly massive, some thin bedding, stylolites.		22.60	1															
24																				
25					2															
26		END OF HOLE		25.80																
27																				
28																				
29																				
30																				
31																				
32																				

DEPTH SCALE

1:50



LOGGED: PKS

CHECKED: LCC

MISS. ROCK 1143-ROCK.GPJ GLDR. CAN.GDT 28/5/03 PS/MMZ

PROJECT 021-1143			RECORD OF BOREHOLE No 20-3			1 OF 1			METRIC														
W.P. 275-99-00			LOCATION N 4769677.3; E 326361.1			ORIGINATED BY PKS																	
DIST 4 HWY 406			BOREHOLE TYPE Solid Stem Augers			COMPILED BY DKB																	
DATUM Geodetic			DATE October 25, 2002			CHECKED BY ASP																	
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES																		
183.2	GROUND SURFACE																						
0.0	Topsoil		1	SS	4																		
0.2	Silty Clay, trace sand and gravel, occasional oxidation staining, trace rootlets, fissured Firm to hard Brown/grey Moist		2	SS	22																		
			3	SS	28																		
			4	SS	28																		
			5	SS	77																		
179.1	Clayey Silt, trace sand and gravel, occasional silty clay layers Very stiff Brown to reddish grey Wet																						
4.1			6	SS	27																		
178.2	Clayey Silt, trace sand and gravel Very stiff Reddish grey Wet																						
5.0			7	SS	26																		
176.5	END OF BOREHOLE																						
6.7	Note: 1. Open borehole dry upon completion of drilling.																						

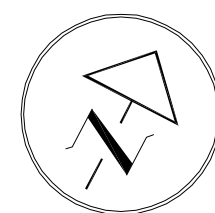
MISS MTO 021-1143.GPJ ON MOT.GDT 27/5/03

PROJECT <u>021-1143</u>		RECORD OF BOREHOLE No HF-12		1 OF 1	METRIC
W.P. <u>275-99-00</u>	LOCATION <u>N 4769721.9; E 326418.9</u>	ORIGINATED BY <u>PKS</u>			
DIST <u>4</u> HWY <u>406</u>	BOREHOLE TYPE <u>Solid Stem Augers</u>	COMPILED BY <u>DKB</u>			
DATUM <u>Geodetic</u>	DATE <u>October 30, 2002</u>	CHECKED BY <u>ASP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
183.3	GROUND SURFACE													
183.3	Topsoil Silty Clay, trace sand and gravel, occasional oxidation staining, fissured Very stiff to hard Brown/grey Moist		1	SS	16									
			2	SS	33									
			3	SS	32									
			4	SS	29									
			5	SS	32									
179.2	Clayey Silt, trace sand and gravel, occasional silt layers Very stiff Reddish grey to grey Moist		6	SS	19									
			7	SS	28									
176.6	END OF BOREHOLE													
6.7	Note: 1. Base of open borehole wet upon completion of drilling.													

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

DIST. 4 HWY. 406
CONT No.
WP No. 275-99-00



HIGHWAY 406 / REGIONAL
ROAD 20 UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



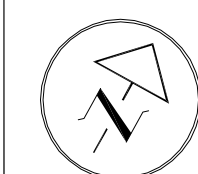
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



KEY PLAN



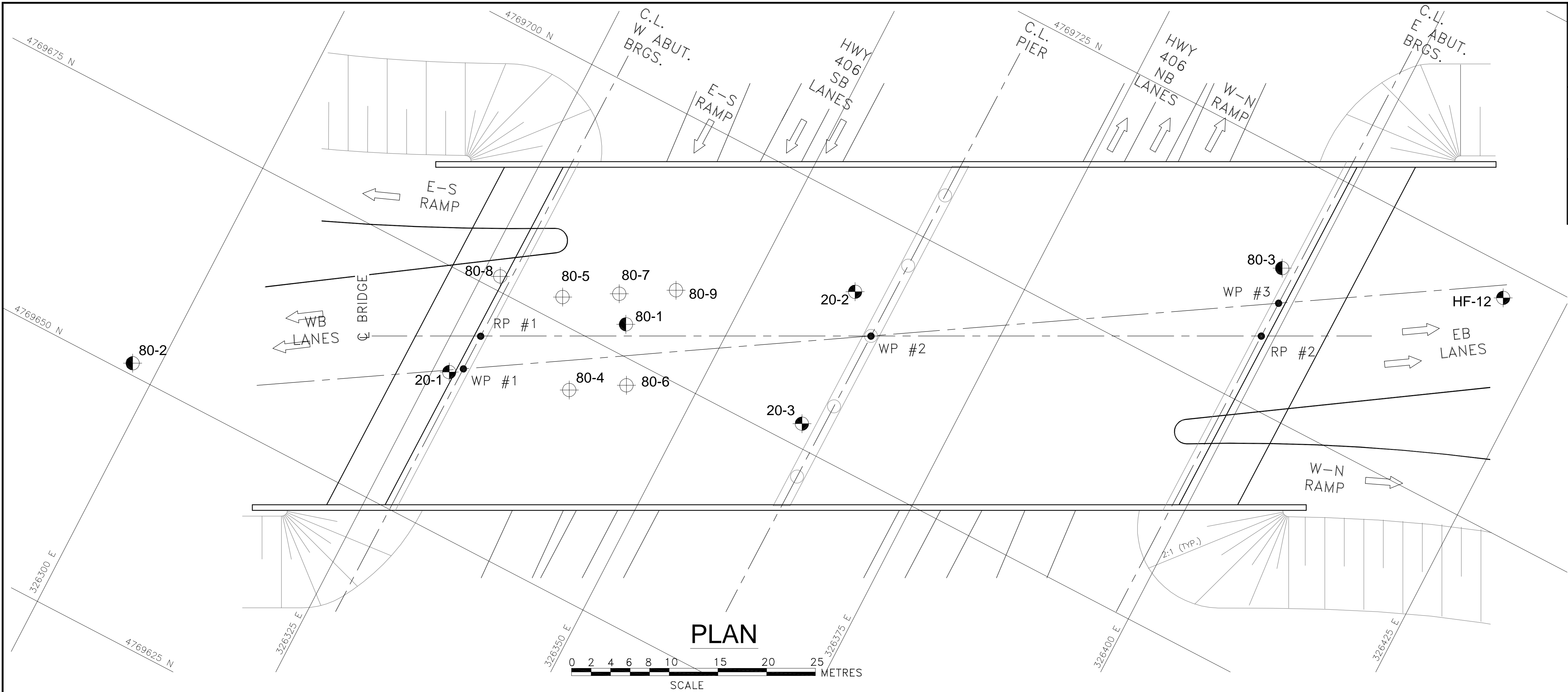
LEGEND

- Borehole - Current Golder Associates Ltd. Investigation
- Borehole - Previous MTO Investigation (Geocres No. 30M3-175, dated Aug. 1980)
- Probehole - Previous MTO Investigation (Geocres No. 30M3-175, dated Aug. 1980)
- 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 January 31, 2003
- WL upon completion of drilling for current Boreholes and 7 days after drilling for previous Boreholes

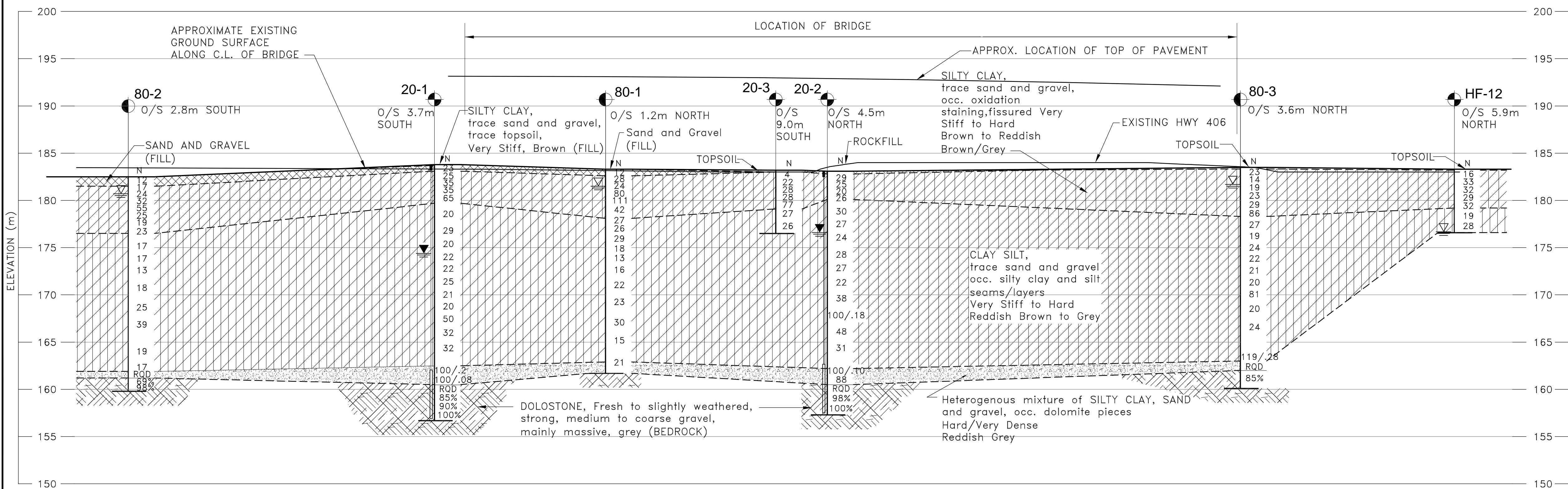
No.	ELEVATION	LOCATION	
		NORTHING	EASTING
20-1	183.8	4769665.3	326326.6
20-2	183.8	4769691.8	326359.7
20-3	183.2	4769677.3	326361.1
HF-12	183.3	4769721.9	326418.9
80-1	183.3	4769678.0	326340.4
80-2	183.5	4769651.1	326297.4
80-3	183.5	4769714.1	326397.4
80-4	183.5	4769669.4	326338.4
80-5	183.7	4769677.5	326333.4
80-6	183.2	4769672.5	326343.3
80-7	183.5	4769680.5	326338.4
80-8	184.0	4769676.4	326326.7
80-9	183.2	4769683.5	326343.3

NOTES

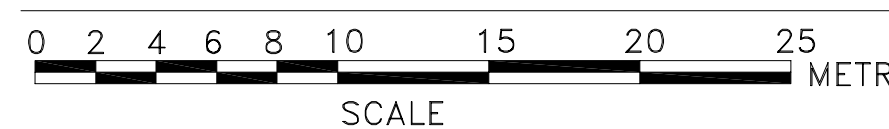
The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.



PLAN



CENTRELINE PROFILE



REFERENCE

Base Plan provided in digital format from Morrison Hershfield Ltd. File No. ROAD20-S01.dwg, received Dec. 9, 2002

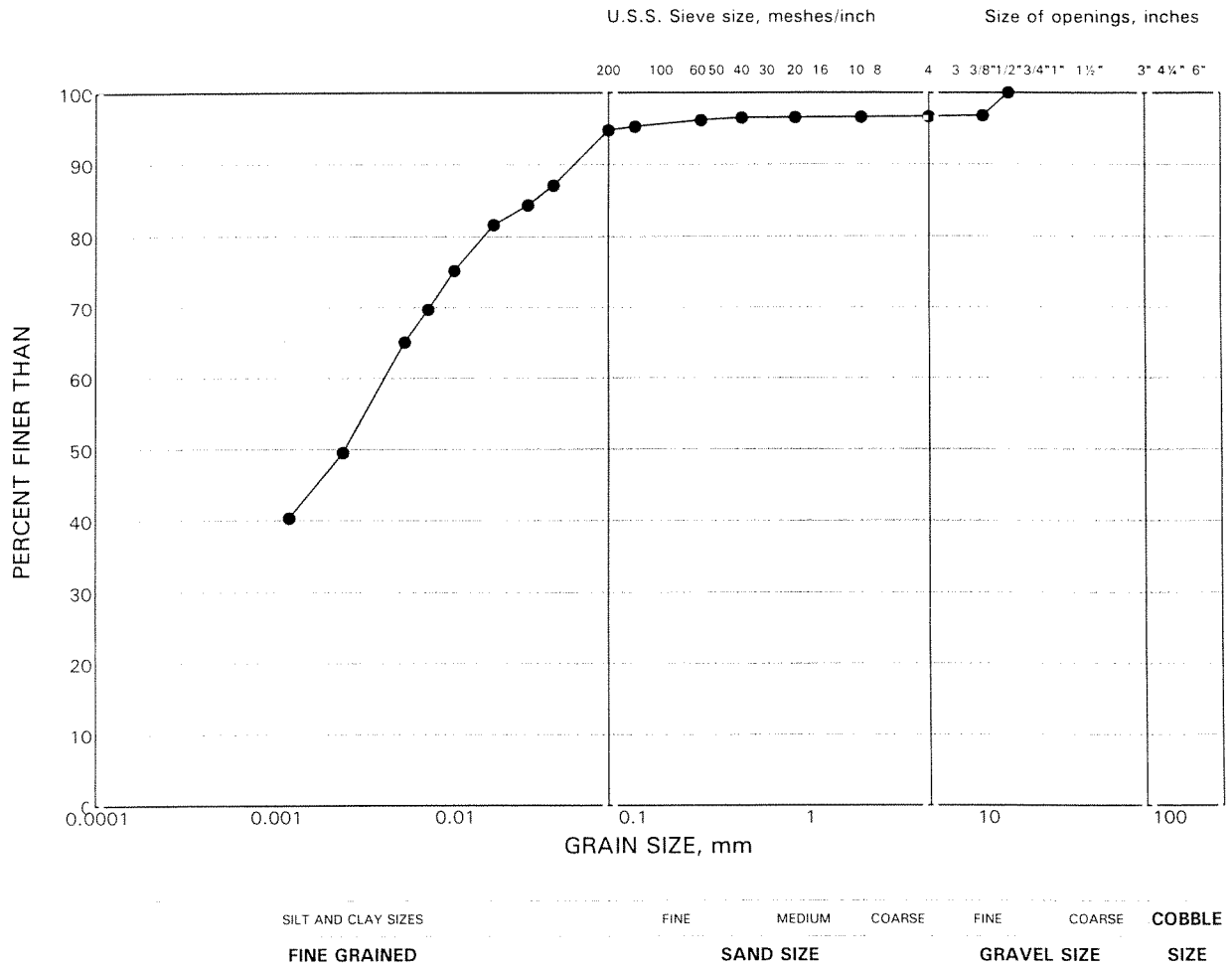
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

NO.	DATE	BY	REVISION
HWY. 406 (TWINNING)			PROJECT NO. 021-1143-1
SUBM'D. DKB	CHKD. DKB	DATE: APRIL 2003	SITE:
DRAWN: JFC	CHKD. ASP	APPD. FJH	DWG. 1

GRAIN SIZE DISTRIBUTION

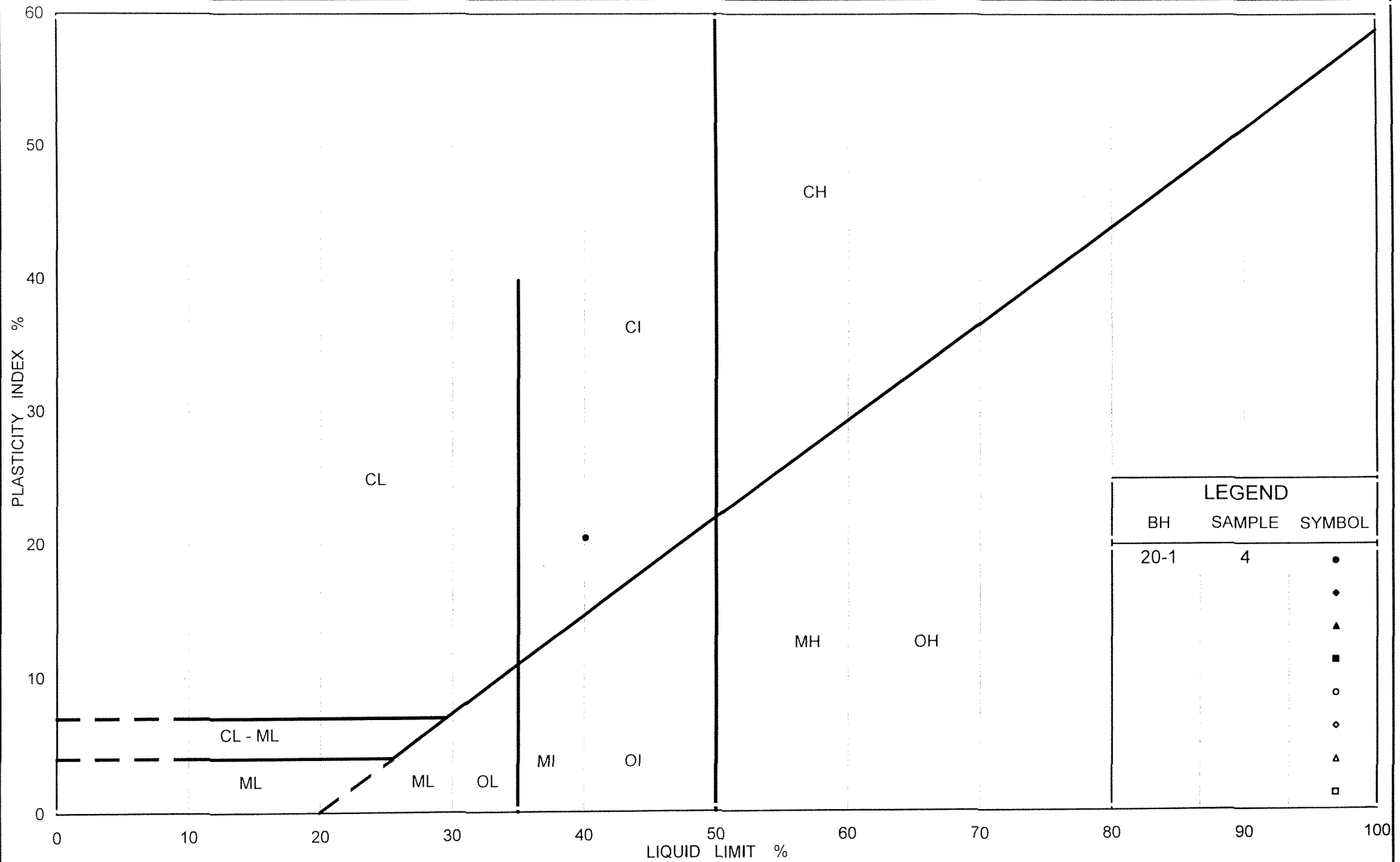
Silty Clay, trace sand and gravel

FIGURE 1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	20-1	4	180.9



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay

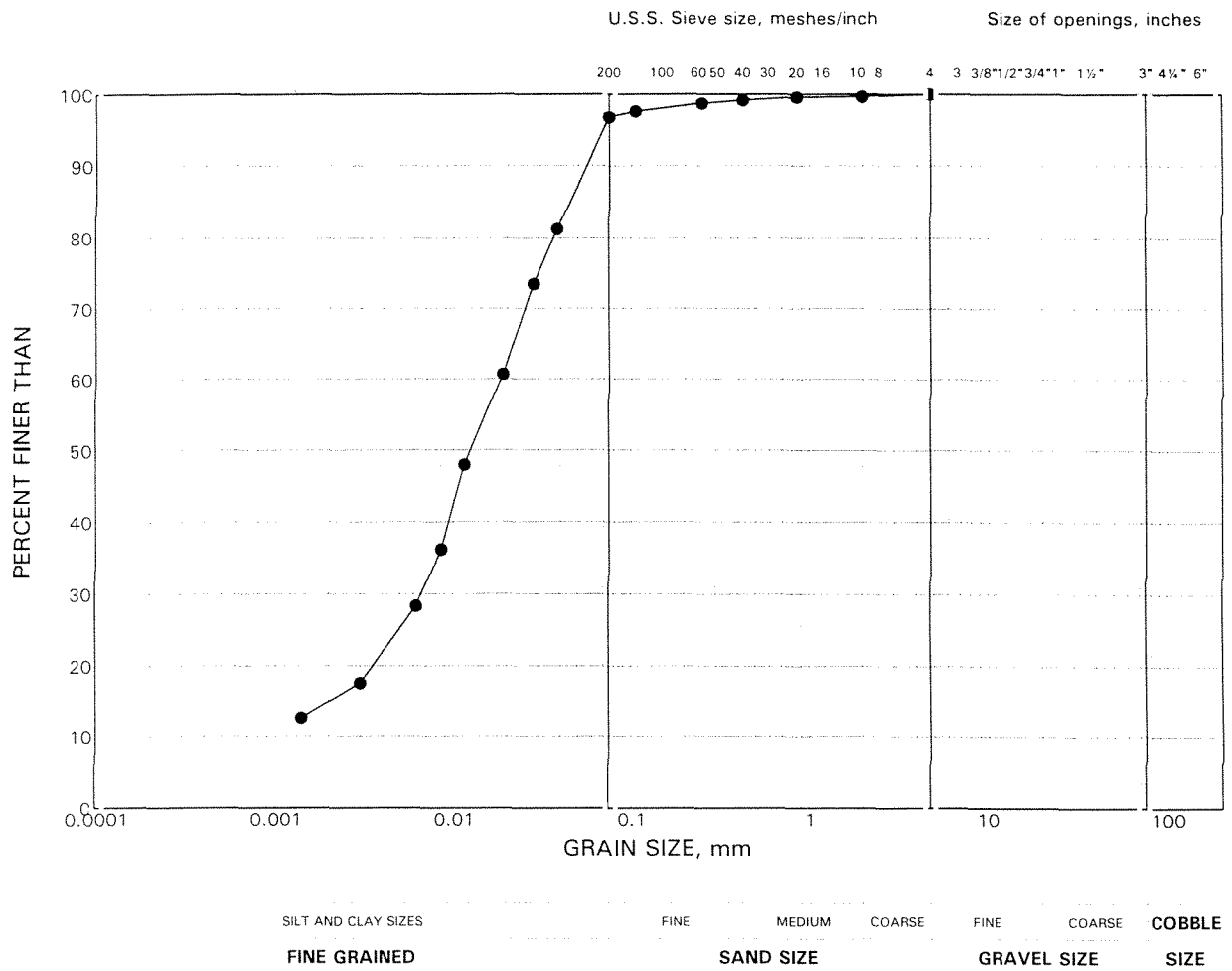
FIG No. 2

Project No. 021-1143-1

GRAIN SIZE DISTRIBUTION

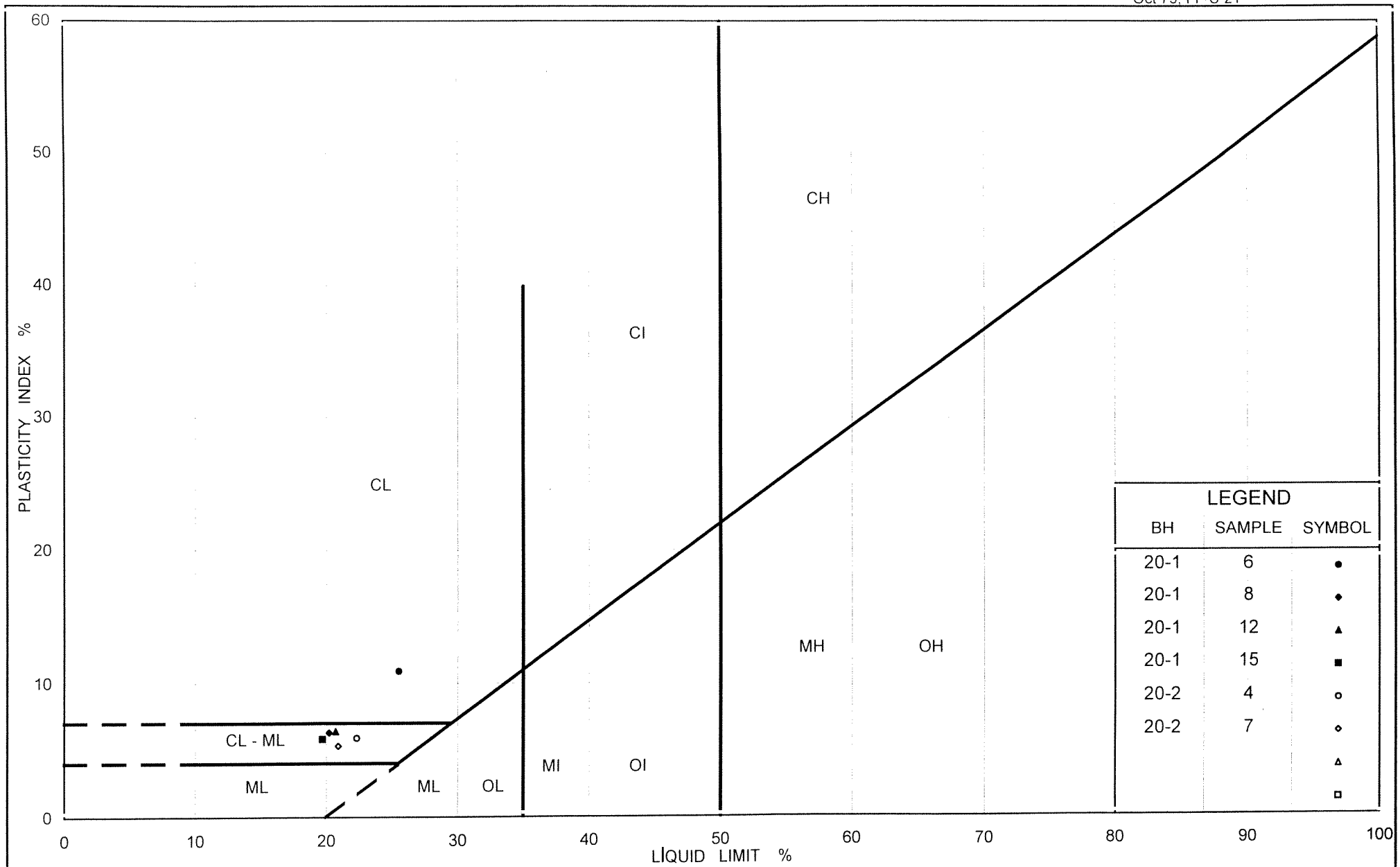
Clayey Silt, trace sand

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	20-1	12	169.5



Ministry of Transportation

Ontario

PLASTICITY CHART Clayey Silt

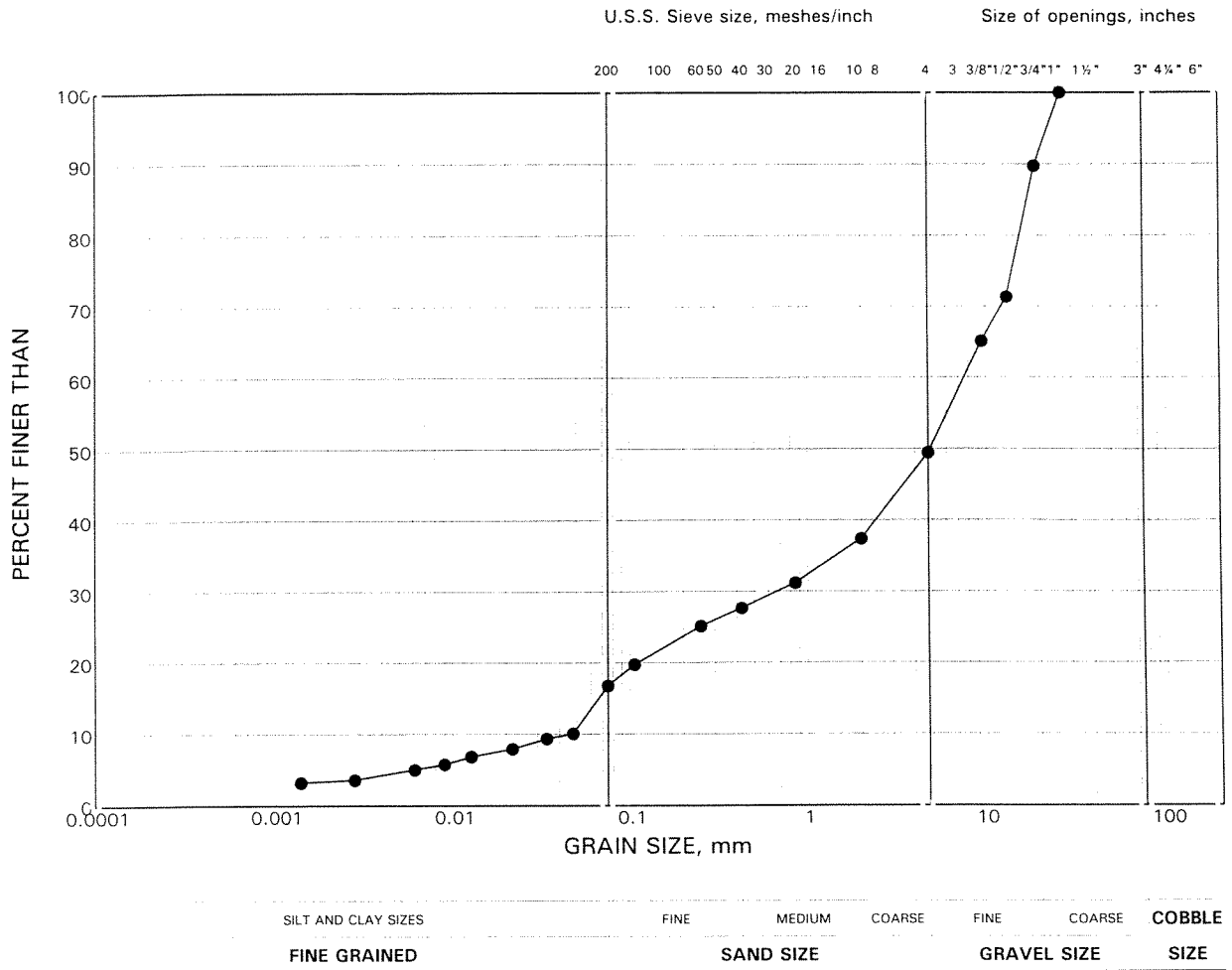
FIG No. 4

Project No. 021-1143-1

GRAIN SIZE DISTRIBUTION

Sand and Gravel, some silt, trace clay

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	20-2	16	161.2

APPENDIX A

**RELEVANT RECORD OF BOREHOLE SHEETS
(MTO GEOCRES NO 30M3-175, DATED AUGUST 1980)**

W P 88-63-00 LOCATION Co-ords N 4 769 463 E 326 341 ORIGINATED BY SC
DIST 4 HWY 406620 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SC
DATUM Geodetic DATE June 3, 1980 CHECKED BY _____

* 3, x⁵: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 2

W P 88-63-00 LOCATION Co-ords N 4 769 436 E 326 298 ORIGINATED BY SC
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SC
 DATUM Geodetic DATE June 4, 1980 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH						
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE					
183.5														
0.0	Roadway Fill													
182.5	(Sand and Gravel)		1	SS	12									
1.0	Silty Clay		2	SS	17									
	(Low to Medium Plasticity)		3	SS	24									
	Traces of Sand and Gravel		4	SS	32									
	Very Stiff to Hard		5	SS	55									
			6	SS	25									
177.5			7	SS	19									
6.0	Layers of Silty Clay (of Low Plasticity) and Silt		8	SS	23									
	Trace of Sand		9	SS	17									
	Stiff to Hard		10	SS	17									
			11	SS	13									
			12	SS	18									
			13	SS	25									
			14	SS	39									
			15	SS	19									
162.9			16	SS	17									
20.6	Het. Mixture of Silty Clay, Sand & Gravel		17	RC	69%									
162.2			18	RC	98%									
21.3	Dolstone Bedrock Slightly Pitted													
160.8														
22.7	End of Borehole													

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity 20 15 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

W P 88-63-00 LOCATION Co-ords. N 4 769 499; E 326 398 ORIGINATED BY SC
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SC
 DATUM Geodetic DATE June 5, 1980 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p		NATURAL MOISTURE CONTENT W		LIQUID LIMIT W _L		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
183.5	Ground Level																
0.0	Topsoil																
	Silty Clay		1	SS	23		182										9 7 42 42
	(Low to Medium Plasticity)		2	SS	14												
	Traces of Sand and Gravel		3	SS	19												
	Stiff to Hard		4	SS	23		180										
			5	SS	29												
178.3			6	SS	86			120/250 mm									
5.2			7	SS	27		178										
	Layers of Silty Clay (of Low Plasticity) and Silt		8	SS	19												
			9	SS	24												
	Trace of Sand						176										
	Very Stiff to Hard		10	SS	22												
							174										
			11	SS	21												
			12	SS	20		172										
			13	SS	81		170										0 1 91 8
			14	SS	20												
							168										
			15	SS	24		166										
							164										
163.0																	
20.5	Het. Mixture of Silty Clay, Sand and Gravel		16	SS	119/	280 mm	162										41 16 34 9
162.0	Hard																
21.5	Dolstone Bedrock		17	RC	85%												
	Sound						160										
160.1																	
23.4	End of Borehole																

+3, x5: Numbers refer to Sensitivity
 20
 15 5 (%) STRAIN AT FAILURE
 10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4

W P 88-63-00 LOCATION Co-ords. N 4 769 456; E 326 338 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM Geodetic DATE June 11, 1980 CHECKED BY _____

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT	PLOT	NUMBER								
183.5 0.0	Ground Level											GR SA SI CL
	Probable Silty Clay											
180.5 3.0	End of Cone Test						120/200 mm					

RECORD OF BOREHOLE No 5

W P 88-63-00 LOCATION Co-ords. N 4 769 464; E 326 333 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM Geodetic DATE June 11, 1980 CHECKED BY _____

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT	PLOT	NUMBER								
183.7 0.0	Ground Level											GR SA SI CL
	Probable Silty Clay											
180.7 3.0	End of Cone Test						120/230 mm					

RECORD OF BOREHOLE No 6

W P 88-63-00 LOCATION Co-ords. N 4 769 459; E 326 343 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM Geodetic DATE June 11, 1980 CHECKED BY _____

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT	PLOT	NUMBER								
183.2 0.0	Ground Level											GR SA SI CL
	Probable Silty Clay											
180.2 3.0	End of Cone Test											

RECORD OF BOREHOLE No 7

88-63-00 LOCATION Co-ords. N 4 769 459; E 326 343 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM Geodetic DATE June 11, 1980 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
183.5 0.0	Ground Level									
	Probable Silty Clay									
180.5 3.0	End of Cone Test									

RECORD OF BOREHOLE No 8

Phone note 80-8

W P 88-63-00 LOCATION Co-ords. N 4 769 467; E 326 338 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM DATE CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
184.0 0.0	Ground Level									
	Probable Silty Clay									
181.0 3.0	End of Cone Test									

RECORD OF BOREHOLE No 9

Phone note 80-9

W P 88-63-00 LOCATION Co-ords. N 4 769 470; E 326 343 ORIGINATED BY PP
 DIST 4 HWY 406 & 20 BOREHOLE TYPE Cone Penetration Only COMPILED BY SC
 DATUM Geodetic DATE June 11, 1980 CHECKED BY

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
183.2 0.0	Ground Level									
	Probable Silty Clay									
180.5 2.7	End of Cone Test									