

**Golder Associates Ltd.**

2390 Argentia Road  
Mississauga, Ontario, Canada L5N 5Z7  
Telephone: (905) 567-4444  
Fax: (905) 567-6561



**PRELIMINARY FOUNDATION  
INVESTIGATION AND DESIGN REPORT  
DOANE ROAD UNDERPASS  
HIGHWAY 404 EXTENSION  
FROM GREEN LANE TO HIGHWAY 12/48  
AGREEMENT NO. 2005-A-000585**

Submitted to:

URS Canada Inc.  
75 Commerce Valley Drive East  
Thornhill, Ontario  
L3T 7N9

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## **PART A**

**PRELIMINARY  
FOUNDATION INVESTIGATION REPORT  
DOANE ROAD UNDERPASS  
HIGHWAY 404 EXTENSION  
FROM GREEN LANE TO HIGHWAY 12/48  
AGREEMENT NO. 2005-A-000585**

## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out preliminary foundation investigations associated with the extension of Highway 404 from Herald Road (Green Lane) in the Regional Municipality of York, to Highway 12/48 in the Regional Municipality of Durham.

Foundation investigation services are required for the following planning and preliminary design components of the Highway 404 extension study:

- **Planning Component:** To satisfy the requirements of the Canadian Environmental Assessment (CEA), foundation investigation is required at three environmentally significant water crossings along the extension, namely Maskinonge River, Black River, and Pepperlaw Brook.
- **Preliminary Design Component:** To satisfy the requirements of the Provincial Class “B” Environmental Assessment, preliminary foundation investigations are required for six proposed road crossings along the Highway 404 extension; from south to north, these are Mt. Albert Road, Doane Road, Queensville Sideroad, Bradford Bypass, Boag Sideroad, and Woodbine Avenue.

This report addresses the preliminary foundation investigation carried out for the Doane Road underpass structure.

The terms of reference for the foundation investigation are outlined in MTO’s Request for Proposal for Agreement No. 2005-A-000585, issued in September 2003, in Clarifications 1 through 9 issued by MTO throughout the proposal preparation period, and in Golder’s proposal which is documented under Sections 4.5 and 5.8 in URS’s *Technical and Management Proposal for the Highway 404 Extension*.

## **2.0 SITE DESCRIPTION**

The proposed Doane Road underpass structure is located approximately 750 m west of the existing Doane Road – Woodbine Avenue (York Regional Road 8) intersection, and approximately 75 m south of the existing Doane Road alignment, in the Town of East Gwillimbury in the Regional Municipality of York. The area south and north of Doane Road is currently occupied by farmland.

The overall surface topography in the Town of East Gwillimbury is generally flat-lying to gently sloping; drumlins – elliptical “hills” formed by advancing glaciers during the last period of glaciation – are present throughout the area. The proposed structure site is located at the south end of a drumlin, near the base of its eastern flank. The natural ground surface in the immediate area of the structure site is at about Elevation 261 m to 259 m, sloping downward toward the east and south away from the drumlin. To the north of the existing Doane Road alignment, the drumlin has a maximum ground surface elevation of approximately 280 m.

### 3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the proposed Doane Road underpass structure in September 2004, at which time three boreholes (Boreholes 201 to 203) were advanced at the site using a track-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario.

The boreholes were advanced using hollow stem augers to depths ranging from 10.9 m to 19.9 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open boreholes was observed throughout the drilling operations, and a piezometer was installed in Borehole 203 to monitor the groundwater level at the site. Boreholes 201 and 202 were backfilled to ground surface using bentonite pellets upon completion. In Borehole 203, the piezometer tip and sand filter pack were backfilled to ground surface using bentonite pellets; this installation will be abandoned in accordance with Regulation 903 requirements once additional groundwater level readings are obtained.

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

The borehole locations were measured by Golder Associates relative to site features, and the ground surface elevations at the borehole locations were determined from the DTM for this project. The borehole locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

<b><i>Borehole Number</i></b>	<b><i>MTM NAD83 Northing (m)</i></b>	<b><i>MTM NAD83 Easting (m)</i></b>	<b><i>Ground Surface Elevation</i></b>
201	4,887,071.1	310,371.6	261.6 m
202	4,887,088.0	310,393.7	261.0 m
203	4,887,106.0	310,419.2	260.7 m

## 4.0 SITE GEOLOGY AND STRATIGRAPHY

### 4.1 Regional Geological Conditions

The study area for this assignment lies within three physiographic regions, as delineated in *The Physiography of Southern Ontario*<sup>1</sup>:

- The Schomberg Clay Plain, which is present from the existing northern terminus of Highway 404 at Herald Road (Green Lane) to south of Mt. Albert Road.
- The Peterborough Drumlin Field, which is present at two locations along the proposed alignment: from south of Mt. Albert Road to approximately Ravenshoe Road (just north of the proposed Woodbine Avenue structure); and again from about Pepperlaw Road (near the boundary of York and Durham Regions) to beyond the eastern limit of the proposed extension at Highway 12/48.
- The Simcoe Lowlands, which are present along the proposed alignment from approximately Ravenshoe Road to Pepperlaw Road.

The Doane Road underpass site is located within the Peterborough Drumlin Field physiographic region.

The surficial soils in the Schomberg Clay Plain consist of stratified deposits of clay and silt, with averages thickness of about 4 m to 5 m, that overlie a drumlinized till plain (which is contiguous with the Peterborough Drumlin Field). The rolling relief of the underlying till plain has not been entirely eliminated, and so this region is not as flat as typical lake plains. Most of the smaller drumlins have been completely covered by silts and clays; however, some of the larger drumlins in this region are not completely buried.

The surficial soils in the Peterborough Drumlin Field consist of drumlinized till. Toward the western portion of this physiographic region, where the Highway 404 extension will be constructed, the till is typically sandy. Some of the drumlins in this area have shallow coverings of silt and fine sand, between about 0.5 m and 2.5 m in thickness. “Wave-washed” drumlins, with exposed bouldery surfaces, are also present near the Simcoe Lowlands immediately south and east of Lake Simcoe. Localized deposits of silt, clay and peat are found in the low-lying areas between drumlins.

The surficial soils in the Simcoe Lowlands, to the south and southeast of Lake Simcoe, consist of sands, silts and clays that were deposited within a former glacial lake. Throughout Georgina Township, where much of the Highway 404 extension will be constructed, the plain is generally

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



low and swampy; Black River and Pepperlaw Brook, the most important streams in this area, have failed to provide good drainage, and generally occupy swampy valleys. It is noted that several areas of drumlinized till break the continuity of the Simcoe Lowlands plain. Such areas, which formed islands in the former glacial lake, occur along the proposed highway extension near Keswick and Belhaven, and again in the vicinity of Virginia.

## **4.2 Site Stratigraphy**

As part of the subsurface investigation at this site, three boreholes were advanced at the site in the vicinity of the proposed overpass structures. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 7. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

Surficial deposits, consisting of a thin layer of compact sand and silt overlying 1.5 m to 4 m of firm to stiff clayey silt, are present at the site. These surficial deposits overlie a sequence of glacial and interglacial soils, consisting of two compact to very dense / hard till layers, separated by a compact to very dense layer of silty sand to sand and gravel; a lower layer of very dense silty sand was encountered below the lower till in one of the boreholes. Based on the borehole results, the deposits typically become thicker and the deposit surfaces decline toward the east, away from the drumlin. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and a stratigraphic section is provided on Drawing 2.

### **4.2.1 Topsoil**

Approximately 200 mm of topsoil was encountered in each of the boreholes.

### **4.2.2 Surficial Sand and Silt**

A 0.5 m to 1.3 m thick upper layer of sand and silt, containing trace gravel, is present in all of the boreholes immediately underlying the topsoil. The measured Standard Penetration Test (SPT) “N” values within the upper sand and silt range from 7 to 14 blows per 0.3 m of penetration, indicating that the layer has a loose to compact relative density.

### **4.2.3 Surficial Clayey Silt**

A surficial clayey silt deposit was encountered in all of the boreholes, underlying the upper sand and silt layer. This clayey silt ranges in thickness from 1.5 m to 4 m, with its base encountered in the boreholes between 2.2 m and 5.5 m depth (Elevation 259.4 m to 255.2 m), declining eastward.

The surficial clayey silt contains trace sand; silt seams were observed within the deposit in one of the boreholes. The results of two grain size distribution tests are shown on Figure 1. Atterberg limit tests were carried out on four samples of the upper clayey silt deposit, and measured plastic limits of 15 to 20 per cent, liquid limits of 21 to 25 per cent, and plasticity indices of 5 to 6 per cent. The results, plotted on Figure 2, confirm that the surficial clayey silt deposit has a low plasticity. The water contents measured on samples of the clayey silt are near the liquid limit for the material.

The measured SPT “N” values within the clayey silt range from 4 to 15 blows per 0.3 m of penetration. One field vane test was carried out; it measured an undrained shear strength of approximately 54 kPa and a remoulded shear strength of about 10 kPa. These results indicate that the surficial clayey silt has a firm to stiff consistency and is sensitive.

### **4.2.4 Upper Sand and Silt Till**

A 1.2 m to 4.7 m thick upper glacial till deposit is present below the surficial clayey silt stratum. The surface of the upper glacial till was encountered in the boreholes between 2.2 m and 5.5 m depth (Elevation 259.4 m to 255.2 m), declining eastward.

The glacial till consists of sand and silt, containing trace to some clay and trace gravel; the result of one grain size distribution test on this material is shown on Figure 3. An Atterberg limit test was carried out on one sample of the upper sand and silt till, and measured a plastic limit of 11 per cent, and a plasticity index of slightly less than 4 per cent. The result, plotted on Figure 4, confirms that the material is principally a silt of very low plasticity.

The measured SPT “N” values range from 17 to 86 blows per 0.3 m of penetration. The lowest values of 17 and 24 blows were measured within the shallowest, thinnest portion of the till, as encountered at the west end of the structure site in Borehole 201; the till in this area has a compact relative density. The SPT “N” values in Boreholes 202 and 203, where the till deposit becomes deeper and thicker, were 34 to over 50 blows per 0.3 m of penetration, indicative of a dense to very dense relative density.

#### **4.2.5 Middle Silty Sand to Sand and Gravel**

A 2.3 m to 3.1 m thick cohesionless deposit was encountered in all three boreholes below the upper till deposit. The cohesionless deposit ranges in composition from silty sand containing trace gravel, to sand and gravel containing trace to some silt; the results of two grain size distribution tests are shown on Figure 5. Layering of the silty sand and sand and gravel was observed in some of the recovered samples.

The SPT “N” values measured within this deposit range from 18 to greater than 100 blows per 0.3 m of penetration, indicative of a variable, compact to very dense relative density.

#### **4.2.6 Lower Sand and Silt Till / Clayey Silt Till**

A lower glacial till deposit underlies the middle silty sand to sand and gravel deposit. The surface of the lower till was encountered between approximately 5.8 m and 13.3 m depth (about Elevation 255.8 m to 247.4 m), declining eastward. The lower till is approximately 2.7 m thick in Borehole 201, where it was fully penetrated; this till is at least 4.5 m to 6.6 m thick as encountered in Boreholes 202 and 203, where it was not fully penetrated.

The lower till is generally comprised of sand and silt containing trace gravel and trace to some clay; however, in Borehole 203, the sand and silt till grades with depth to a clayey silt till containing some sand and trace gravel, as well as sand and gravel lenses. The results of two grain size distribution tests are shown on Figure 6. Atterberg limit tests were carried out on two samples of the lower till deposit: one sample of the sand and silt till was proved to be non-plastic; and one sample of the clayey silt till from Borehole 203 had a plastic limit of 14 per cent, a liquid limit of 28 per cent, and a plasticity index of 14 per cent. The corresponding water content is near the plastic limit for this material. The result of the Atterberg limits test, plotted on Figure 7, confirms that this clayey silt till is of low plasticity.

The sand and silt portion of the lower till has a very dense relative density, and the clayey silt portion of the lower till has a hard consistency, based on measured SPT “N” values that exceed 100 blows per 0.3 m of penetration.

#### **4.2.7 Lower Silty Sand**

The lower silty sand layer was encountered beneath the lower glacial till in Borehole 201, at a depth of approximately 8.5 m (about Elevation 253.1 m); the borehole was terminated within this layer. The measured SPT “N” values within the lower silty sand are greater than 100 blows per 0.3 m of penetration, indicative of a very dense relative density.

### 4.3 Groundwater Conditions

The middle silty sand to sand and gravel and lower silty sand strata are water-bearing, and comprise the main aquifers at the site. The upper sand and silt may also be considered as a minor, unconfined aquifer; although the samples recovered during the drilling investigation were moist (not wet), it should be expected that groundwater could be “perched” within this deposit by the underlying clayey silt during periods of heavy precipitation or snow melt. The upper and lower sand and silt till deposits are also water-bearing, although not highly permeable as the proportion of clay-sized mineral within these deposits is up to about 10 per cent.

A piezometer was sealed into the upper till and middle silty sand deposits in Borehole 203 to monitor the groundwater level at the site. The following table summarizes the water level readings measured in the piezometer since completion of the borehole:

<i>Borehole No.</i>	<i>October 1, 2004</i>		<i>October 7, 2004</i>	
	<i>Depth</i>	<i>Elevation</i>	<i>Depth</i>	<i>Elevation</i>
203	1.1 m	259.6 m	1.2 m	259.5 m

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

## **5.0 CLOSURE**

This Preliminary Foundation Investigation Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

### **GOLDER ASSOCIATES LTD.**

Lisa C. Coyne, P.Eng.  
Associate

Fintan J. Heffernan, P.Eng.  
Designated MTO Foundations Contact

BC/LCC/FJH/lcc

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**PART B**

**PRELIMINARY  
FOUNDATION DESIGN REPORT  
DOANE ROAD UNDERPASS  
HIGHWAY 404 EXTENSION  
FROM GREEN LANE TO HIGHWAY 12/48  
AGREEMENT NO. 2005-A-000585**

## **6.0 PRELIMINARY ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides foundation design recommendations for the preliminary design of the proposed Doane Road underpass structure. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out preliminary design of the proposed structure foundations. Where comments are made on construction they are provided in order to highlight those aspects which could affect the preliminary design of the project.

Further borehole drilling will be required during the detail design phase of the project, when the location and ultimate configuration of the proposed underpass structure is known. At that time, further investigation of the surficial clayey silt will be required to confirm the magnitude and time rate of settlement under the embankment loading, and to further assess the settlement mitigation options and develop the necessary operational constraints and/or special provisions for the contract. In addition, boreholes will be required for the Doane Road cut immediately to the west of the proposed structure; in that area, Doane Road will pass through an existing drumlin.

### **6.2 Bridge Foundation Options**

The proposed Doane Road underpass will be a two-span structure. The natural ground surface in the immediate vicinity of the structure site slopes downward to the south and east, from about Elevation 261.5 m to 259.5 m. It is understood that the finished grade of Highway 404 will be at about Elevation 262 m, and the Doane Road grade will be at about Elevation 270.5 m within the limits of the underpass structure and its immediate approach embankments. As such, the approach embankments will be about 9 m to 11 m high relative to the existing natural ground surface.

The subsoils encountered in the boreholes drilled to date consist of a thin surficial sand and silt layer overlying a 1.5 m to 4 m thick, firm to stiff clayey silt deposit, in turn underlain by a sequence of compact to very dense glacial till and interglacial, cohesionless deposits. The currently proposed alignment of Doane Road is located about 50 m south of Boreholes 201 to 203 and, since the new structure location is located slightly further down the flank of the drumlin, there is potential for the surficial clayey silt deposit to be thicker than that encountered in the boreholes drilled to date.

The firm to stiff, surficial clayey silt deposit is not suitable for support of shallow foundations, and so it is recommended that the foundations be extended below this deposit, either by founding spread footings on the upper till deposit, or driving steel H-piles into the lower till deposit. Consideration

could also be given to adopting spread footings “perched” within the approach embankment, following subexcavation of the surficial clayey silt or a period of preloading / surcharging.

The following foundation options are available, based on consideration of the proposed road grades and the subsurface conditions:

- **Spread footings founded below the surficial clayey silt, on the upper till deposit:** This option is considered suitable for support of the centre pier, where approximately 3.5 m to 4 m of subexcavation is anticipated based on the results of the boreholes advanced during this preliminary foundation investigation. It could also be used to support the abutments although approximately 5.5 m of subexcavation would be required for the east abutment footing, and the resulting abutment walls would be quite high. The new Doane Road alignment is located about 75 m south of the existing roadway alignment within a farm field, and therefore it is anticipated that the excavations could be advanced in open cut, without need for temporary roadway protection.
- **Spread footings “perched” within the approach embankments on a granular pad:** The loading imposed by the 9 m to 11 m high approach embankments will cause approximately 25 mm to 60 mm of consolidation settlement within the surficial clayey silt deposit, and a further 5 mm to 15 mm of compression within the cohesionless soil deposits. For “perched” footings to be feasible, it would be necessary to preload (and potentially surcharge) the area by constructing the approach embankments in advance; alternatively, the surficial clayey silt stratum could be subexcavated.
- **Steel H-piles driven to found within the very dense lower till deposit:** Driven steel H-piles are suitable for support of the abutments (in either a conventional or integral abutment configuration) and centre pier. The loading imposed by the approach embankments will cause settlement of the surficial clayey silt deposit and overlying embankment fill, which will in turn impart downdrag loads on the piles. It is recommended that the approach embankments for the Doane Road structure be constructed in advance, in order to eliminate the downdrag forces on the piles. Since the new Doane Road alignment is located within a farm field approximately 75 m south of the existing road, there is sufficient space and there would be no interruption to traffic flow on the local road during construction.

Recommendations for preliminary design of spread footings and steel H-pile foundations are presented in the following sections. A summary comparison of the advantages, disadvantages and relative costs associated with each of the feasible foundation options is presented in Table 1 following the text of this report. From a foundations perspective, based on this comparison, it is considered that driven steel H-piles or “perched” abutment footings are the most practicable foundation solutions for this site. In both cases, it will be necessary to construct the approach embankments in advance to allow for approximately one year of preloading.



## **6.3 Spread Footings**

### **6.3.1 Spread Footings Founded on Upper Till Deposit**

The foundation elements could be supported on spread footings placed below the surficial clayey silt deposit, on the compact to very dense upper till deposit. It is noted that about 3 m to 5.5 m of subexcavation will be required, based on the results of the boreholes advanced at the site as part of the preliminary foundation investigation. For preliminary design purposes, a footing founding level of about Elevation 259 m to 255 m should be assumed, generally declining from west to east; further investigation will be required during detailed design to confirm these founding levels. In addition, the final ditching grades should be checked to ensure that a minimum of 1.5 m of soil cover (or equivalent) is provided above the footing level for frost protection purposes.

For preliminary design, a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa may be used for 3 m wide spread footings placed on the surface of the properly prepared, compact to very dense upper till deposit. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 450 kPa for preliminary design. It is noted, however, that the ULS resistance and settlement are dependent on the footing size, configuration and applied loads. The geotechnical resistances should, therefore, be reviewed during detailed design, once further drilling has been carried out at the foundation elements to delineate the thickness of the surficial clayey silt and confirm the founding level, and once the final geometry of the foundations has been established.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination 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 curves for non-cohesive soils.

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,  $\tan \phi'$ , between cast-in-place concrete footings and the undisturbed, properly prepared upper sand and silt till may be taken as 0.5. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

### 6.3.2 Spread Footings “Perched” Within Approach Embankments

In order to minimize the height of the abutment walls, spread footings for the bridge abutments may be placed on a compacted Granular “A” pad constructed within the approach embankment fill. For “perched” spread footings to be a viable alternative at this site, one of the following options will be necessary:

**Option 1:** Construct the approach embankments, including the Granular “A” pad, approximately six months to one year in advance of construction of the footings and bridge structure, to allow the majority of the consolidation settlement of the surficial clayey silt to occur. Further evaluation of the time rate of settlement will be required during detailed design to assess the necessary preloading duration, and settlement monitoring during the preloading period is recommended.

*or*

**Option 2:** Subexcavate the surficial clayey silt deposit, to minimize the settlement due to the embankment loading. It is expected that the subexcavation will extend to between 3 m and 5.5 m depth, based on the results of the boreholes advanced as part of the preliminary investigation; however, further investigation will be required during detailed design to confirm the thickness of the surficial clayey silt within the limits of the abutments and approach embankments. The area to be subexcavated should be defined by a line extending from the toe of the Granular “A” pad, outward and downward at 1 horizontal to 1 vertical (1H:1V). The subexcavation should be replaced with compacted Granular “B”.

Assuming that one of the above options is adopted, a factored geotechnical resistance at ULS of 900 kPa may be used for preliminary design of the “perched” spread footings. The geotechnical resistance at SLS may be taken as 350 kPa, assuming that the Granular “A” and the Granular “B” backfill have a total thickness of at least one footing width. These geotechnical resistances will have to be reviewed during detailed design, after further drilling has been carried out at the foundation elements to confirm the extent of subexcavation that is required, and once the final geometry of the foundations and approach embankments has been established.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *CHBDC* and its *Commentary*, using the curves for non-cohesive soils.

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,  $\tan \delta$ , between cast-in-place concrete footings and the compacted Granular “A” pad may be taken as 0.57. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

#### 6.4 Steel H-Pile Foundations

The abutments may be supported on steel H-piles driven to found within the “100-blow” lower till deposit or, where present, the very dense lower silty sand stratum. The surface of the lower till was encountered between 5.8 m and 13.3 m depth (about Elevation 255.8 m to 247.4 m) in the boreholes. It is noted that further borehole investigation will be required within the limits of the proposed foundation elements to confirm the pile tip elevations. However, in order to achieve a minimum factored axial resistance at ULS of 1,300 kN, the following tip elevations may be assumed for preliminary design of HP 310 x 110 piles:

<i>Foundation Element</i>	<i>Design Pile Tip Elevation</i>	<i>Approximate Pile Length*</i>
West Abutment	254 m	15 m
Centre Pier	251 m	8 m
East Abutment	245 m	24 m

\* Approximate pile lengths given assuming that the abutment pile caps are perched within the approach embankments, with a minimum of 1.5 m of soil cover (or equivalent) to provide adequate protection against frost penetration. The abutment pile cap underside is assumed to be at or below about Elevation 269 m, and the centre pier pile cap underside is assumed to be at or below about Elevation 259 m.

The settlement of the individual piles and the pile group at the above loads is anticipated to be less than 25 mm. Therefore, the geotechnical resistance at SLS may be taken as 1,300 kN.

As noted in Section 6.2, the approach embankment loading will result in between 25 mm and 60 mm of post-construction consolidation settlement within the surficial clayey silt deposit, with consequent settlement of the overlying embankment fill. This settlement will result in negative skin friction loads on the abutment piles. For preliminary design, an unfactored downdrag load acting on a single pile of 500 kN will need to be taken into account in the design of the abutment piles; the structural capacity of the piles must be checked for the factored dead and downdrag loads, in accordance with Section 6.8.4 of the *CHBDC*. Consideration should be given to the construction of the approach embankments approximately six months to one year in advance of the bridge construction to allow the majority of the consolidation settlement to occur, thereby eliminating the downdrag loads on the piles.

## 6.5 Retained Soil System (RSS) Walls

If consideration is being given to a false abutment configuration at this site, the retained soil system (RSS) walls would be up to about 8 m high. At this site, subexcavation of the firm to stiff surficial clayey silt will be required, in order to obtain an adequate factor of safety against global instability of the RSS wall. The subexcavation should be carried out to the approximate depths given in the table below, within the area defined by a line extended from the base of the RSS walls outward and downward at a 1H:1V slope. It is noted that further investigation will be required during detailed design to delineate the thickness and consistency of the surficial clayey silt within the limits of the abutments and approach embankments.

<i>Location</i>	<i>Subexcavation Requirements</i>
West abutment	Upper 2 m of soil
East abutment	Upper 5 m of soil

The subexcavation should be backfilled with Granular “B” fill. For RSS walls founded on the compacted Granular “B” fill, a factored geotechnical resistance at ULS of 300 kPa may be used for design. This value assumes 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 geotechnical resistance at SLS is governed by the settlement which will occur due to the combined loading of the RSS wall and approach embankment. It is predicted that up to about 25 mm of settlement will occur due to consolidation of the remaining portion of the surficial clayey silt deposit and compression of the underlying upper till deposit, under the assumed embankment loading of approximately 200 kPa to 220 kPa.

The resistance to lateral forces / sliding resistance between the compacted Granular “A” and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction,  $\tan \delta$ , between the compacted Granular “A” of the RSS wall and the compacted granular fill used following subexcavation may be taken as 0.65. This coefficient of friction 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, provided that subexcavation of the surficial clayey silt deposit is carried out as outlined above, prior to construction of the walls.

## 6.6 Approach Embankment Design and Construction

The construction of the approach embankments at the Doane Road site will require placement of about 9 m to 9.5 m of fill for the west approach, and about 10.5 m to 11 m of fill for the east approach. It should be noted that a mid-height berm, having a minimum width of 2 m, will be required for the approach embankments since they are greater than 8 m in height.

Based on the results of the boreholes drilled to date, the soils within the limits of the approach embankments consist of a firm to stiff surficial clayey silt stratum, overlying a sequence of compact to very dense glacial till and cohesionless interglacial deposits. The surficial clayey silt extends to between 2.2 m and 5.5 m depth (deepening toward the east), as encountered in Boreholes 201 to 203.

### 6.6.1 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 9 m to 11 m high approach embankments with 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 have been carried out based on the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd. Both undrained and effective stress analyses were carried out.

<i>Soil Type</i>	<i>Bulk Unit Weight</i>	<i>Effective Angle of Friction</i>	<i>Undrained Shear Strength</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m <sup>3</sup>	32° to 35°	—
Surficial sand and silt	20 kN/m <sup>3</sup>	30°	—
Surficial clayey silt	20 kN/m <sup>3</sup>	28°	50 kPa
Upper sand and silt till	21 kN/m <sup>3</sup>	32°	—

### 6.6.2 Approach Embankment Settlement

Settlement of the approach embankments at the site will occur due to compression of the new embankment fill itself, as well as consolidation of the surficial clayey silt and some compression of the underlying, upper sand and silt till. Provided that the embankment material consists of select subgrade material or clean earth fill, the settlement of the 9 m to 11 m high approach embankment fill itself is expected to be less than 25 mm. The use of granular fill for the new

embankment construction would minimize this magnitude, since the majority of settlement of granular fills will occur during construction.

The settlement of the founding soils has been estimated using the consolidation parameters and elastic deformation moduli given in the table below, based on correlations with the undrained shear strength, Atterberg limits, and SPT “N” values:

<i>Soil Unit</i>	<i>Bulk Unit Weight</i>	<i>Preconsolidation Pressure</i>	<i>C<sub>c</sub></i>	<i>C<sub>r</sub></i>	<i>Elastic Modulus</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m <sup>3</sup>	—	—	—	—
Loose to compact surficial sand and silt	20 kN/m <sup>3</sup>	—	—	—	15 MPa
Surficial clayey silt	20 kN/m <sup>3</sup>	220 kPa	0.13	0.013	—
Compact to very dense upper till	21 kN/m <sup>3</sup>	—	—	—	50 MPa

It is estimated that the embankment loading will just exceed the preconsolidation pressure of the surficial clayey silt deposit, and that about 25 mm to 60 mm of primary consolidation settlement will occur within this deposit; the majority of this settlement is expected to occur within six months to one year following construction of the approach embankments. A further 5 mm to 15 mm of compression will occur in the loose to compact surficial sand and silt and the compact to very dense upper sand and silt till; the majority of the compression within the surficial sand and upper till will occur during or immediately following construction of the approach embankments. It is noted that the above settlement estimates are for preliminary design purposes only, and further investigation will be required at the detailed design stage.

Based on the above settlement estimates, preloading of the approach embankment area is recommended (as discussed in Sections 6.2 to 6.4).

## **7.0 CLOSURE**

This Preliminary Foundation Design Report was prepared by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

### **GOLDER ASSOCIATES LTD.**

Lisa C. Coyne, P.Eng.  
Associate

Fintan J. Heffernan, P.Eng.  
Designated MTO Contact

BC/LCC/FJH/lcc

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**TABLE 1**  
**COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**  
**DOANE ROAD UNDERPASS STRUCTURE**

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Approximate Costs (See Note)</i>
Spread footings supported on surficial clayey silt	<ul style="list-style-type: none"> <li>Not feasible for support of foundation elements</li> </ul>	<ul style="list-style-type: none"> <li>Not applicable</li> </ul>	<ul style="list-style-type: none"> <li>Not applicable</li> </ul>	<ul style="list-style-type: none"> <li>Not applicable</li> </ul>
Spread footings founded below surficial clayey silt, on upper sand and silt till, or on compacted granular fill following subexcavation of surficial clayey silt	<ul style="list-style-type: none"> <li>Feasible for support of abutments and centre pier</li> </ul>	<ul style="list-style-type: none"> <li>Shorter structure length</li> </ul>	<ul style="list-style-type: none"> <li>Significant subexcavation required</li> </ul>	<ul style="list-style-type: none"> <li>Approximately \$280,000 for abutments and centre pier, assuming 4 m wide footings, plus subexcavation and Granular “B” backfill costs on the order of \$90,000</li> </ul>
Spread footings “perched” on granular pad in approach embankment fill	<ul style="list-style-type: none"> <li>Feasible for support of abutment footings</li> </ul>	<ul style="list-style-type: none"> <li>Subexcavation of surficial clayey silt deposit at abutment locations could be avoided by preloading of approach embankment area; structure site located well south of existing Doane Road alignment, so minimal impact on traffic during preloading period</li> </ul>	<ul style="list-style-type: none"> <li>Schedule would have to accommodate six months to one year of preloading time</li> <li>Longer structure length</li> <li>Subexcavation would still be required to adopt spread footings at centre pier location</li> </ul>	<ul style="list-style-type: none"> <li>Approximately \$280,000 for abutments and centre pier (assuming 4 m wide footings), plus subexcavation and Granular “B” backfill costs at the centre pier on the order of \$30,000</li> <li>This estimate excludes the cost of the compacted granular pads at the abutments</li> </ul>
Steel H-pile foundations driven to found within lower sand and silt till	<ul style="list-style-type: none"> <li>Feasible for support of all foundation elements and concrete retaining walls</li> </ul>	<ul style="list-style-type: none"> <li>Negligible settlement</li> <li>Site conditions appropriate for use of integral abutments</li> <li>Downdrag loads can be mitigated by preloading of approach embankment area; as noted above, minimal impact on traffic during preloading period</li> </ul>	<ul style="list-style-type: none"> <li>Downdrag loads will act on piles due to consolidation of surficial clayey silt under embankment loading; schedule would have to accommodate six months to one year of preloading time</li> <li>If RSS walls (with integral abutments in a false abutment configuration) are under consideration to shorten structure span length, it will be necessary to subexcavate the surficial clayey silt in order to achieve an adequate factor of safety against global instability</li> </ul>	<ul style="list-style-type: none"> <li>Approximately \$325,000 for piles and pile caps, assuming single row of piles at abutments (i.e. integral abutment-type structure) and two rows of battered piles at centre pier</li> <li>This does not include cost for RSS wall construction and materials (if false abutment configuration adopted) and does not reflect cost of potentially longer structure span lengths if open configuration adopted</li> </ul>

**NOTE:** The approximate costs provided in the table above are rough estimates and are intended for comparison purposes only. The costs assume a single two-span underpass structure, with each foundation element approximately 24 m in length to allow for two lanes of traffic in each direction on Doane Road, plus tapers for speed change lanes.



## 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
DO	Drive open
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 cm<sup>2</sup> 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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)
$\gamma$	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

$\pi$	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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

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

#### (b) Hydraulic Properties

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

#### (c) Consolidation (one-dimensional)

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

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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  
 \* density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density x acceleration due to gravity)

PROJECT		04-1111-016		RECORD OF BOREHOLE No BH 201		1 OF 1 METRIC						
W.P.				LOCATION		N 4887071.1 ; E 310371.6						
DIST		Central HWY 404		BOREHOLE TYPE		108 mm I.D. Hollow Stem Augers						
DATUM		Geodetic		DATE		SEPTEMBER 20, 2004						
				ORIGINATED BY		PKS						
				COMPILED BY		DD						
				CHECKED BY		LCC						
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
261.6	GROUND SURFACE											
0.0	Topsoil											
0.2	Sand and Silt, trace gravel		1	SS	10							
260.9	Compact Brown Moist											
0.7	Clayey Silt, trace sand		2	SS	13							
	Stiff Brown Moist to wet											
			3	SS	8							
259.4												
2.2	Sand and Silt, trace gravel, trace clay (TILL)		4	SS	17							
	Compact Brown Moist											
258.2			5	SS	24							
3.4	Silty Sand, trace gravel to Sand and Gravel, trace to some silt											
	Layered Compact to very dense Brown to grey Wet		6	SS	19							
			7	SS	72							
255.8												
5.8	Sand and Silt, trace gravel, trace clay (TILL)		8	SS	100/15							
	Very dense Grey Wet											
			9	SS	100/18							
253.1												
8.5	Silty Sand, trace gravel		10	SS	105/25							
	Very dense Brown to grey Wet											
250.7			11	SS	100/23							
10.9	End of Borehole											
	Note: Water level at 6.1 m depth (Elevation 255.5 m) upon completion of drilling.											

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3</sup>% STRAIN AT FAILURE

MIS-MTO 001 041111016AAMTO.GPJ GAL-MISS.GDT 27/4/06

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

PROJECT <u>04-1111-016</u>		<b>RECORD OF BOREHOLE No BH 203</b>				2 OF 2 <b>METRIC</b>													
W.P. _____		LOCATION <u>N 4887106.0; E 310419.2</u>				ORIGINATED BY <u>PKS</u>													
DIST <u>Central</u> HWY <u>404</u>		BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers</u>				COMPILED BY <u>DD</u>													
DATUM <u>Geodetic</u>		DATE <u>SEPTEMBER 16, 17, 20, 2004</u>				CHECKED BY <u>LCC</u>													
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa											
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> <span>20 40 60 80 100</span> <span>20 40 60 80 100</span> </div> <div style="display: flex; justify-content: space-between;"> <span>○ UNCONFINED</span> <span>+ FIELD VANE</span> </div> <div style="display: flex; justify-content: space-between;"> <span>● QUICK TRIAXIAL</span> <span>× REMOULDED</span> </div>												
244.6	Sand and Silt, trace gravel, trace clay (TILL) Very dense Grey/brown Moist	14	SS	100/18		245													
16.2	Clayey Silt, with sand to some sand, trace gravel, containing sand and gravel lenses (TILL) Hard Grey Moist	15	SS	100/18		244													
						243													
		16	SS	100/18		242													
240.8		17	SS	100/18		241													
19.9	End of Borehole																		
	Notes:  1. Water level at 6.1 m depth (Elevation 254.6 m) upon completion of drilling.  2. Water level in piezometer measured at 1.1 m depth (Elevation 259.6 m) on October 1, 2004, and at 1.2 m depth (Elevation 259.5 m) on December 7, 2004.																		



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

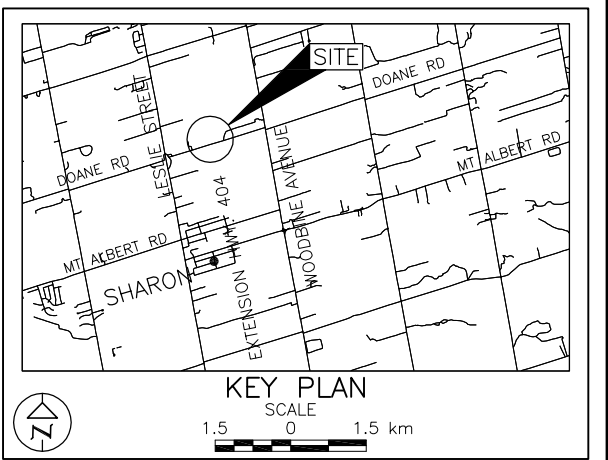
CONT No.  
WP No.

HIGHWAY 404 EXTENSION  
DOANE ROAD STRUCTURE SITE  
BOREHOLE LOCATIONS

SHEET

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole – Current Investigation		
No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
201	261.6	4887071.1	310371.6
202	261.0	4887088.0	310393.7
203	260.7	4887106.0	310419.2

**REFERENCE**


Base plans provided in digital format by URS Canada Inc. Drawing files 404-1997-EA-align-profile.dwg and 404-base-south.dwg received on June 6, 2004, and drawing file X-align-south.dwg received on November 25, 2004.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 404	PROJECT NO. 04-1111-016		DIST.
SUBM'D. PKS	CHKD. LCC	DATE: MAR 2006	SITE:
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1

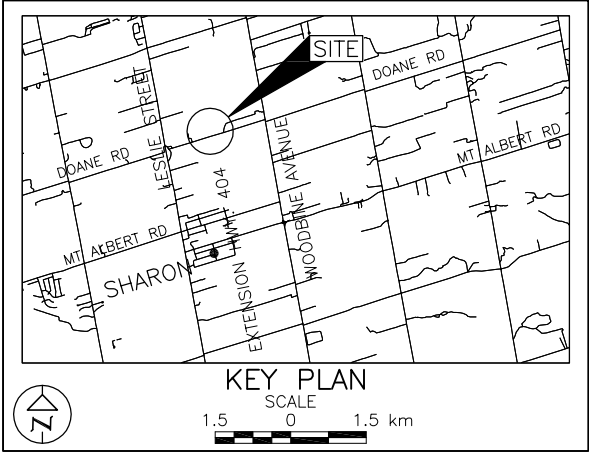
**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.


CONT No.  
WP No.

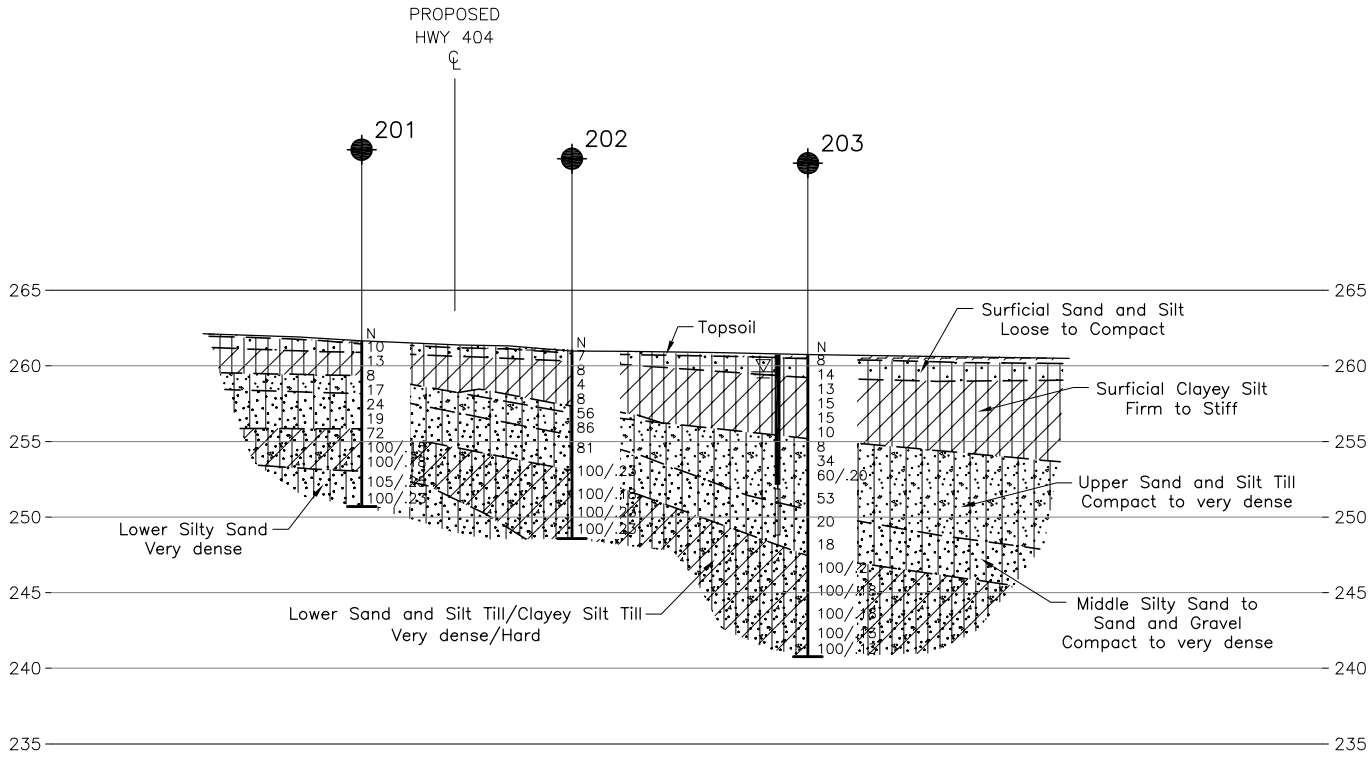
HIGHWAY 404 EXTENSION  
DOANE ROAD STRUCTURE SITE  
SOIL STRATA

**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA

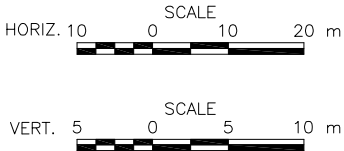
**Golder Associates Ltd.**  
MISSISSAUGA, ONTARIO, CANADA



LEGEND				
	Borehole – Current Investigation			
No.	ELEVATION	CO-ORDINATES		
		NORTHING	EASTING	
201	261.6	4887071.1	310371.6	
202	261.0	4887088.0	310393.7	
203	260.7	4887106.0	310419.2	



**PROFILE ALONG DOANE ROAD**  
A-A'  
1



**NOTES**

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

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

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

**REFERENCE**

Base plans provided in digital format by URS Canada Inc. on June 6, 2004, drawing files 404-base-north.dwg and 404-base-south.dwg. On February 3, 2006, drawings x-align-south.dwg and 404-Doane-profile.dwg

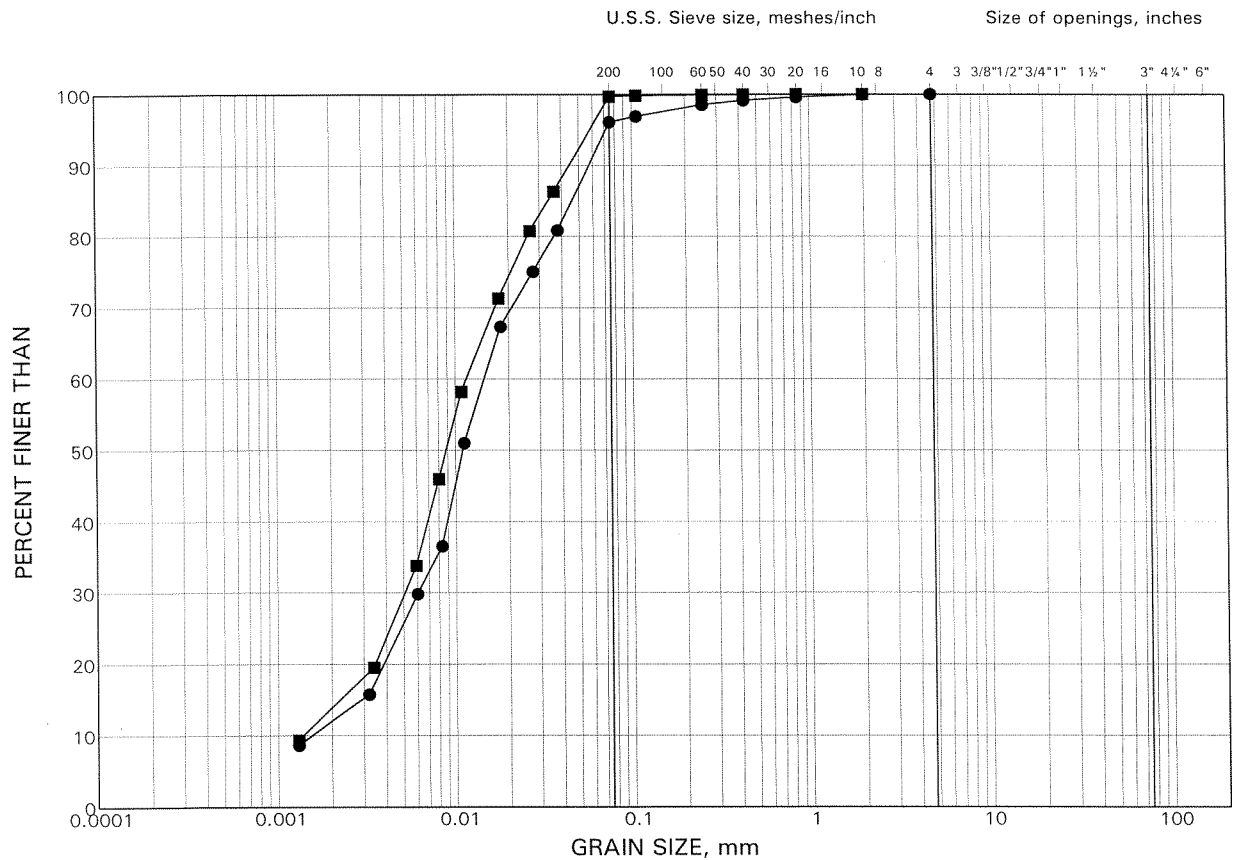
NO.	DATE	BY	REVISION
Geocres No.			
HWY. 404		PROJECT NO. 04-1111-016	DIST.
SUBM'D. PKS	CHKD. LCC	DATE: MAR 2006	SITE:
DRAWN: MSM	CHKD. PKS	APPD. LCC	DWG. 2



# GRAIN SIZE DISTRIBUTION TEST RESULTS

## Surficial Clayey Silt

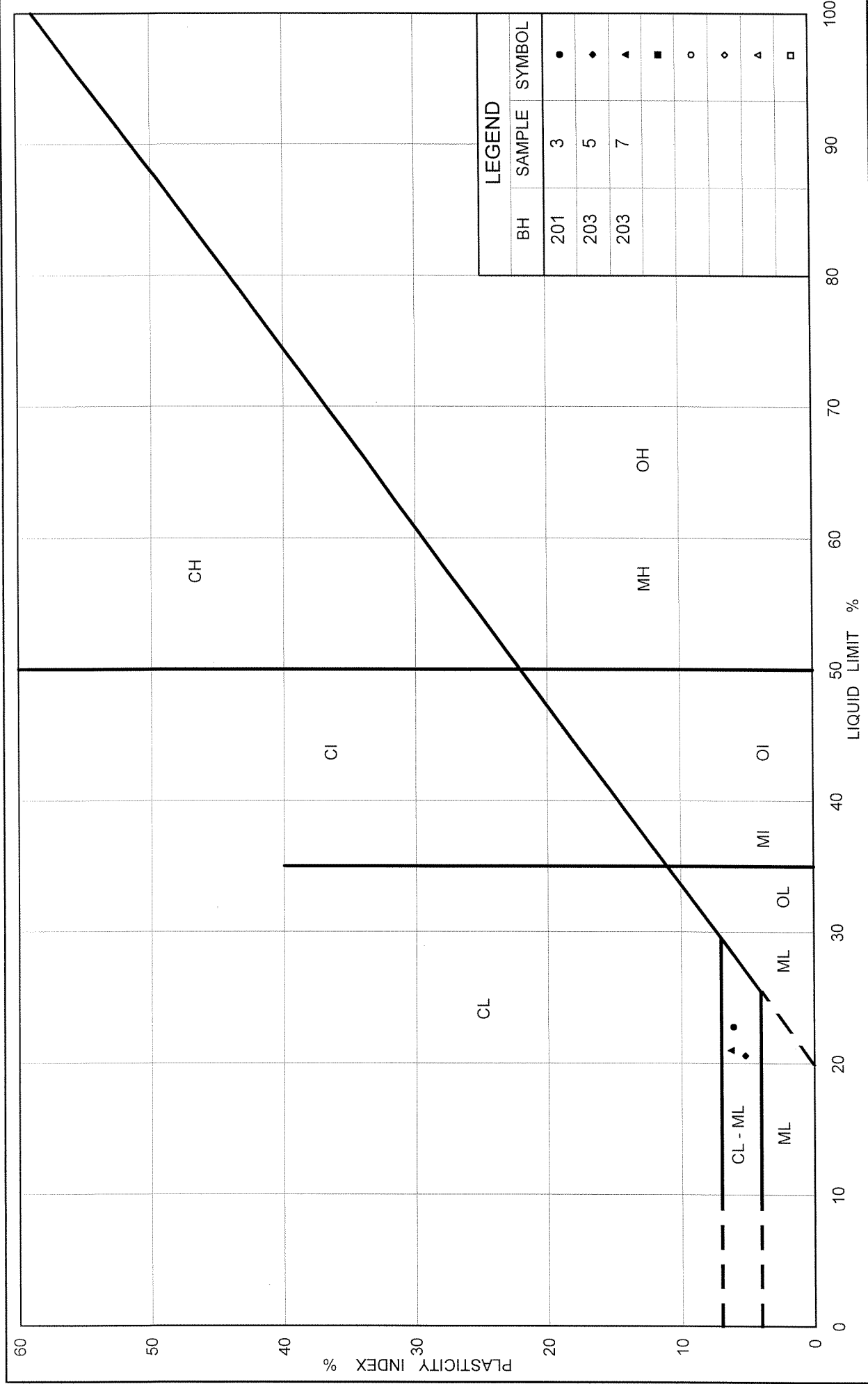
FIGURE 1



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

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	201	3	259.7
■	202	4	258.5



Ministry of Transportation  
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**FIG No. 2**

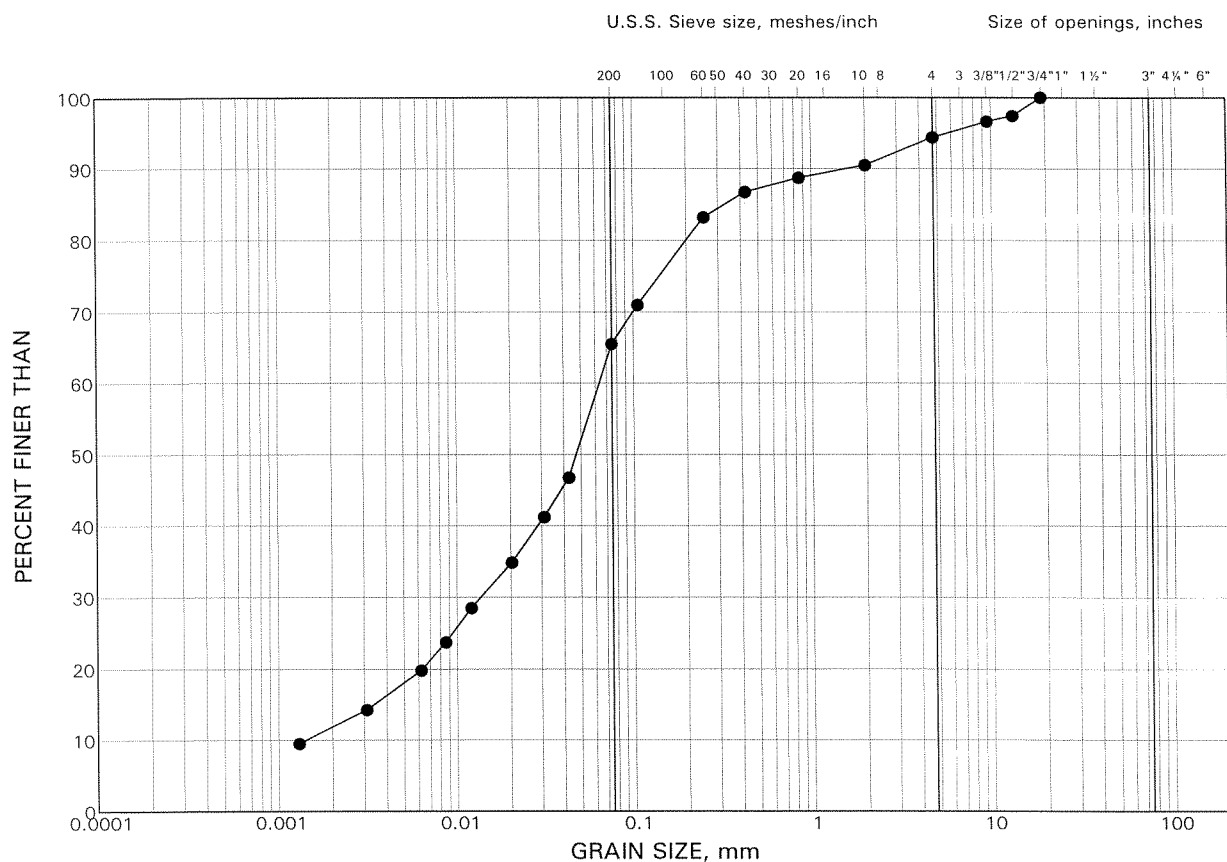
**PLASTICITY CHART**  
Surfacial Clayey Silt

Project No. 04-1111-016

# GRAIN SIZE DISTRIBUTION TEST RESULT

Upper Sand and Silt Till

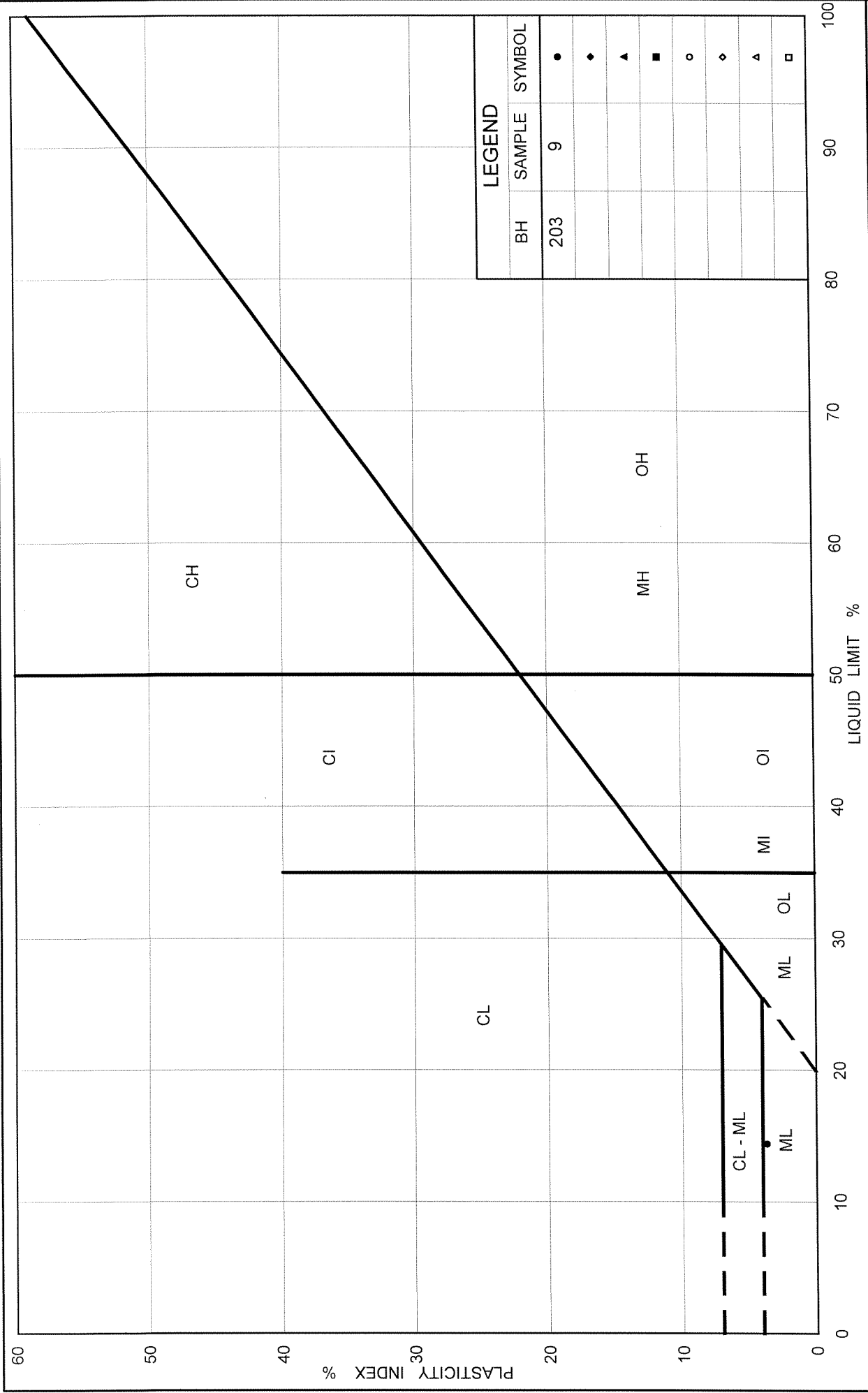
FIGURE 3



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

## LEGEND

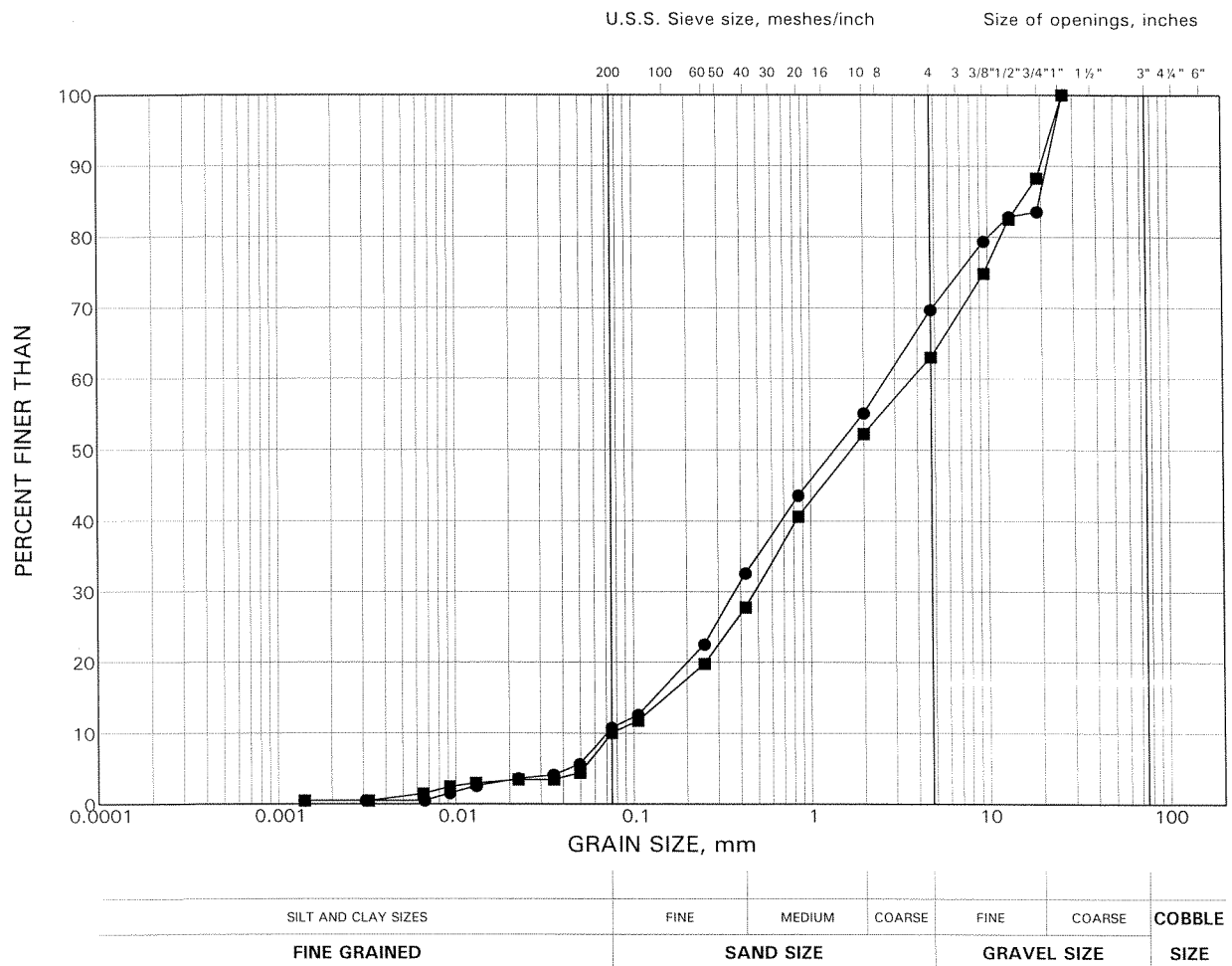
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	203	9	252.9



# GRAIN SIZE DISTRIBUTION TEST RESULTS

Middle Silty Sand to Sand and Gravel

FIGURE 5



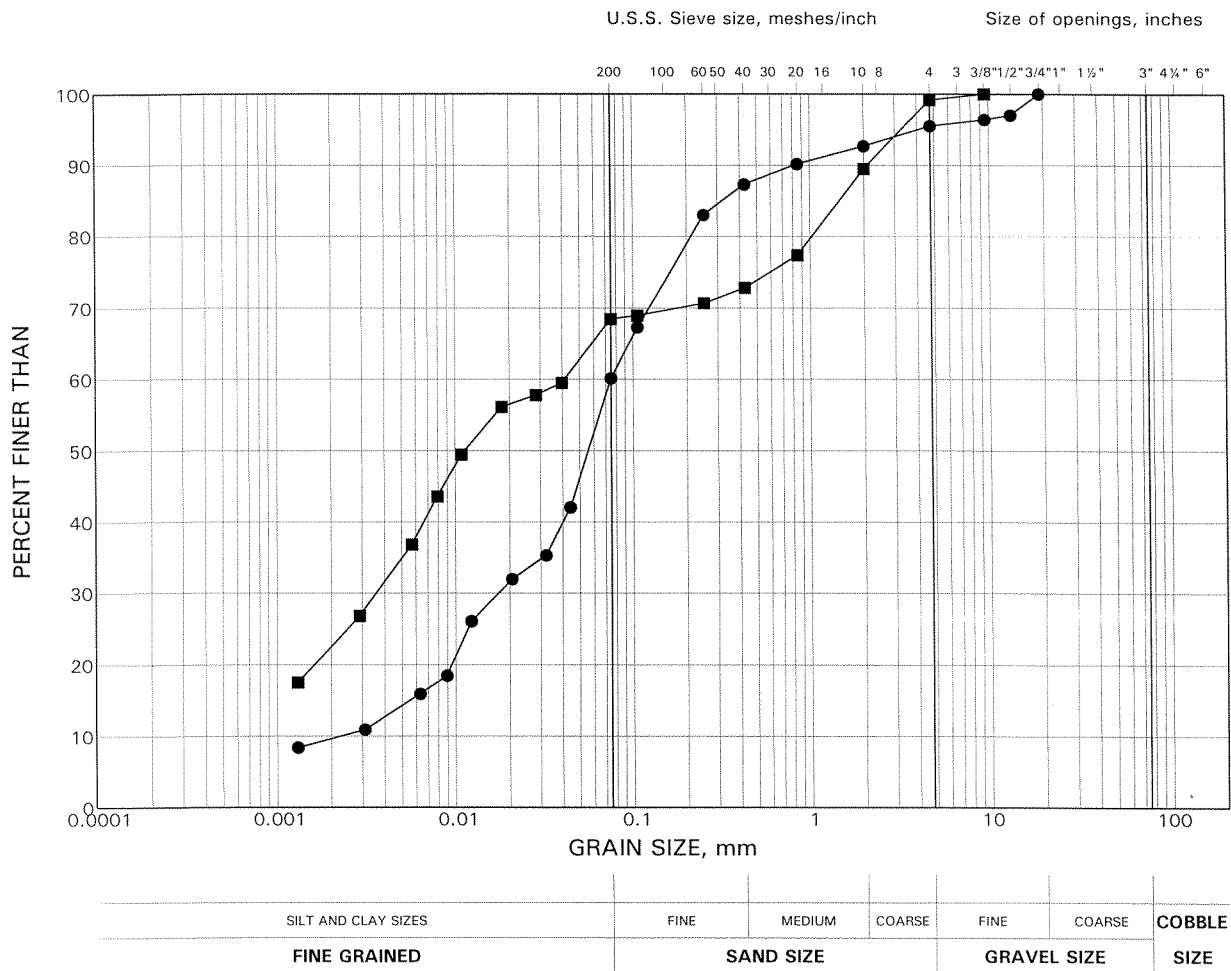
## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	201	6	257.5
■	202	7	254.6

# GRAIN SIZE DISTRIBUTION TEST RESULTS

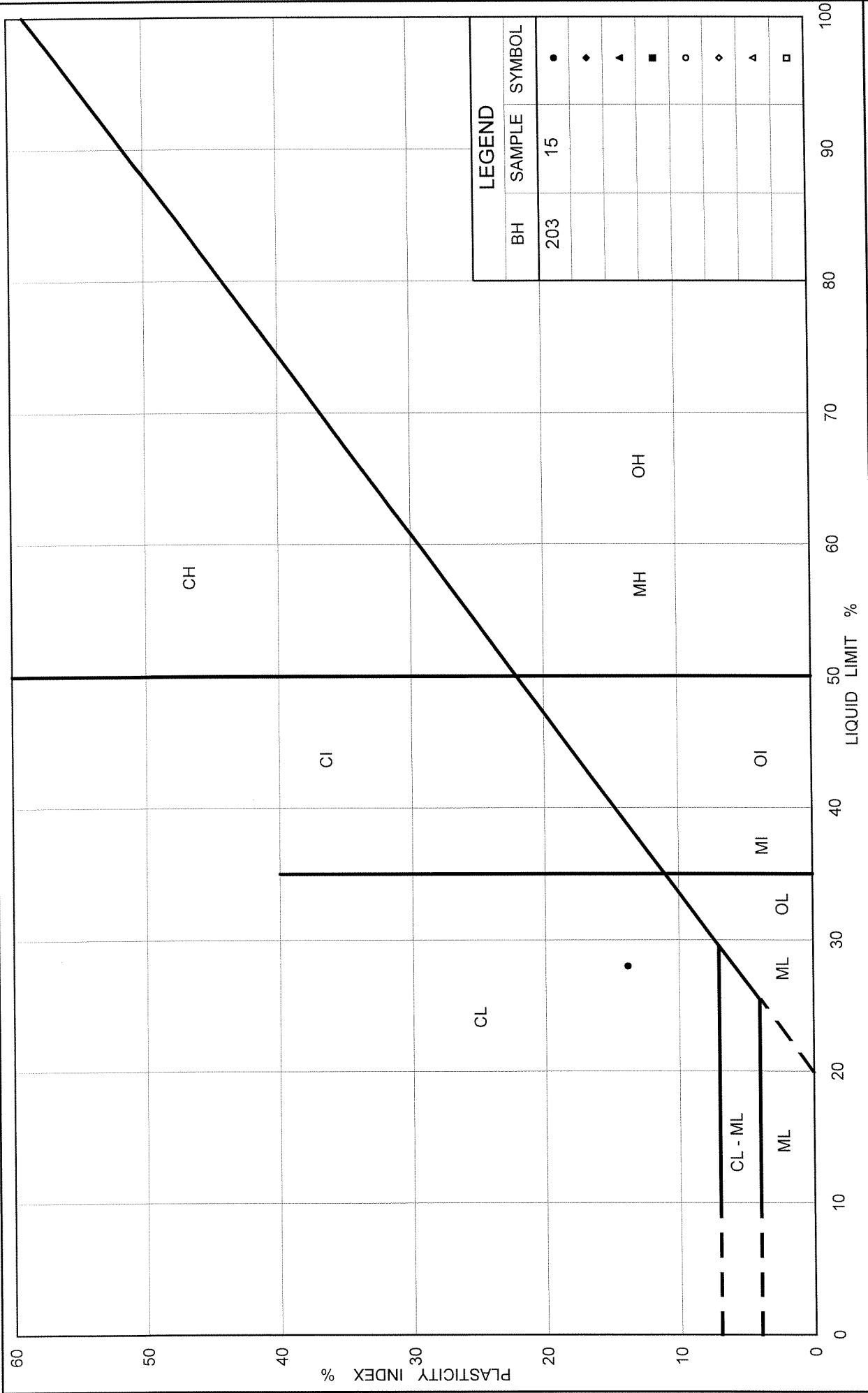
Lower Sand and Silt Till / Clayey Silt Till

FIGURE 6




## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	202	9	251.8
■	203	17	240.9



LEGEND		
BH	SAMPLE	SYMBOL
203	15	•
		◆
		▲
		■
		○
		◇
		△
		□



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PLASTICITY CHART  
Lower Clayey Silt Till

FIG No. 7

Project No. 04-1111-016