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**FOUNDATION
INVESTIGATION AND DESIGN REPORT
ZELLENS ROAD OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING
BETWEEN HIGHWAYS 403 AND 5
G.W.P. 19-95-05**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
ZELLENS ROAD OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
G.W.P. 19-95-05**

1.0 INTRODUCTION

Golder Associates Ltd. has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for new bridge structures, a pedestrian tunnel, culverts, retaining walls, high fill embankments, high mast light poles, and overhead signs, associated with the widening of Highway 6 between Highways 403 and 5 near Dundas, Ontario.

This report addresses the new Zellens Road overhead structure at the CP Rail line. A foundation investigation has been carried out to determine the subsurface conditions at the site.

The terms of reference for the scope of work are outlined in Golder Associates' Proposal No. P01-1166, dated June 2000. The work has been carried out in accordance with Golder Associates' Quality Control Plan for Foundation Engineering Services, dated July 2000.

2.0 SITE DESCRIPTION

This 2.5 km length of Highway 6, between Highway 403 and Highway 5 (Dundas Street), is located within the City of Burlington in the Regional Municipality of Halton, and the Towns of Dundas and Flamborough in the New City of Hamilton.

Highway 6 crosses the Niagara escarpment south of Highway 5, in the vicinity of Old Guelph Road. The escarpment crest is at about Elevation 215 m; above the crest, the ground surface rises northward to about Elevation 220 m near the north limit of the project at Highway 5. The cut through the escarpment – the “Clappison Cut” – was first constructed in 1921. Above Old Guelph Road, near-vertical rock cuts up to about 15 m in height have been constructed on either side of Highway 6. Below the crest, the ground surface declines from Elevation 215 m to about Elevation 147 m in the vicinity of York Road, and about Elevation 133 m near the south limit of the project. Immediately south of Old Guelph Road, Highway 6 has been constructed on embankment fill which is up to about 15 m in height.

At the proposed Zellens Road structure site, the CP Rail grade is at approximately Elevation 133.5 m, rising eastward. The natural ground surface rises northward, from about Elevation 133 m to 133.5 m immediately south of the rail, to about 134 m to 134.5 m immediately north of the rail. Ditches, approximately 0.5 m to 1 m deep, are present along both sides of the rail line.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the Zellens Road overhead structure in November 2004 at which time six boreholes (Borehole Z-3 to Z-8) were advanced in the vicinity of the proposed abutments and immediate approach embankments. The locations of the boreholes are shown on Drawing 1.

Boreholes Z-4 to Z-7 were advanced at the abutment locations to depths of about 11 m to 15m below the existing ground surface, in order to extend through existing fill (where present) and glacial till, and into the underlying hard or very dense clayey silt to sand and silt till / residual soil deposit. Boreholes Z-3 and Z-8 were advanced within the limits of the immediate approach embankments to depths of about 6 m to 7 m below the existing ground surface.

All of the boreholes were advanced by solid stem augers using a track-mounted drill rig supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. Samples of the overburden were obtained at 0.75 m and 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers 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 Z-3 to monitor the groundwater level at the site.

The field work was supervised on a full-time basis by a member 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. Index and classification tests consisting of water content determinations, Atterberg limit testing and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and ground surface elevations were determined relative to points staked in the field by Callon Dietz, Ontario Land Surveyors. The MTM NAD83 northing and easting coordinates for the boreholes, and the ground surface elevations referenced to geodetic datum, are summarized in the following table, and are shown on the borehole records and on Drawings 1 and 2.

<i>Borehole No.</i>	<i>Ground Surface Elevation</i>	<i>MTM NAD83 Northing</i>	<i>MTM NAD83 Easting</i>
Z-3	133.2	4,795,210.7	272,172.3
Z-4	134.4	4,795,214.6	272,149.7
Z-5	133.4	4,795,237.4	272,165.0
Z-6	133.5	4,795,223.1	272,141.1
Z-7	133.5	4,795,246.1	272,156.3
Z-8	134.3	4,795,252.7	272,129.4

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

This 2.5 km section of Highway 6 traverses the Niagara Escarpment, which separates the lower Iroquois Plain to the south from the Flamborough Plain to the north, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, Third Edition, 1984). In the vicinity of the Escarpment itself, covering much of the study area for this project, the Halton Till of the Peel Plain physiographic region is present, according to the *Urban Geology of Canadian Cities* (Karrow and White, 1998).

The escarpment crest is located just north of Old Guelph Road, and well-jointed and bedded sedimentary bedrock consisting of dolostone, limestone, sandstone and shale is exposed in the existing Highway 6 cut. Typically, natural talus intermixed with rubbly glacial debris covers the lower slopes of the escarpment. Below the escarpment, the bedrock consists of shale.

The Halton Till of the Peel Plain physiographic region typically ranges in composition from a dense, reddish clayey silt to silt till to a grey, plastic clayey silt to silty clay till. This Halton Till is the lowest and oldest soil deposit encountered in excavations in the area north of Hamilton, and it typically rests directly on the bedrock. Commonly, there is a transition zone of residual soil and/or disturbed bedrock at the contact between the Halton Till and the shale.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the Record of Boreholes and on Figures 1 to 4 following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

Stratigraphic profiles at the Zellens Road overhead structure site are shown on Drawings 1 and 2. In general, the subsoils at the site consist of thin topsoil and, at some locations, existing fill overlying a generally very stiff to hard clayey silt to silty clay till deposit. The till is underlain at about 4 m to 5.5 m depth by a till / residual soil deposit, that varies from a hard clayey silt to a very dense sandy silt. Shale bedrock was encountered below the till / residual soil deposit in Borehole Z-5, at 13.7 m depth (approximately Elevation 119.7 m).

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

4.2.1 Topsoil

About 100 mm to 250 mm of topsoil was encountered immediately below ground surface in all of the boreholes.

4.2.2 Fill

About 0.6 m and 0.7 m of fill material was encountered immediately below the topsoil in Boreholes Z-4 and Z-5. This fill consists of clayey silt, containing some sand and trace gravel; trace quantities of organics were also noted within one recovered sample.

Standard Penetration Test (SPT) “N” values of 12 blows per 0.3 m of penetration were measured in both boreholes, indicating that this fill has a stiff consistency.

4.2.3 Surficial Clayey Silt

Approximately 0.5 m of brown to dark brown, surficial clayey silt was encountered in Borehole Z-6, immediately below the topsoil layer at this location. This surficial clayey silt contains some sand and trace gravel, and has a firm consistency, based on one measured SPT “N” value of 8 blows per 0.3 m of penetration.

4.2.4 Clayey Silt to Silty Clay Till

A deposit of brown to red-brown till was encountered in all of the boreholes below the topsoil and, where present, the fill and surficial clayey silt. This till deposit is 2.8 m to 4.9 m thick as encountered in the boreholes.

The till is typically comprised of clayey silt containing trace to some sand, and trace gravel, shale and limestone fragments. However, the upper 1 m to 2 m of the till in Boreholes Z-6 and Z-7 is more plastic, based on visual observation and the results of Atterberg testing (discussed below); the upper 1 m to 2 m of the till at this location is, therefore, classified as a silty clay till. The results of grain size distribution tests conducted on two samples of the clayey silt till are shown on Figure 1. It is noted that the till is glacially derived and should, therefore, be expected to contain cobbles and boulders, although no such obstructions were encountered within the till in the boreholes advanced as part of this investigation.

Atterberg limit testing was carried out on six samples of the till, and the results are plotted on a plasticity chart on Figure 2. This testing generally measured plastic limits of 16 to 17 per cent, liquid limits of 26 to 32 per cent, and plasticity indices of 10 to 15 per cent, which confirms that the majority of the till is a clayey silt of low plasticity. The limit testing on a sample of till from Borehole Z-6 measured a plastic limit of 20 per cent, a liquid limit of 41 per cent, and a plasticity index of 21 per cent; these results confirm that the upper 1 m to 2 m of material at the locations of Boreholes Z-6 and Z-7 is a silty clay of intermediate plasticity.

The clayey silt to silty clay till generally has a very stiff to hard consistency, based on measured SPT “N” values of 23 to 74 blows per 0.3 m of penetration (but typically 23 to 50 blows). Lower measured SPT “N” values of 6, 9 and 14 blows per 0.3 m of penetration were measured immediately below ground surface in Boreholes Z-3, Z-7 and Z-8, indicating that the upper layer of the till in these boreholes has a firm to stiff consistency.

4.2.5 Till / Residual Soil

The till deposit grades with depth to a till / residual soil deposit. The surface of the till / residual soil was encountered in the boreholes between 3.7 m and 5.6 m depth, at about Elevation 127.9 m to 130.6 m. The till / residual soil was proved to be about 10 m thick where it was fully penetrated in Borehole Z-5.

The till / residual soil varies in composition from a red-brown or grey clayey silt containing trace to some sand, and trace to some gravel, shale and limestone fragments, to a red-brown or grey sandy silt containing trace clay and trace to some gravel, shale and limestone fragments. Silt seams were observed within some of the recovered samples of the clayey silt till / residual soil. The results of grain size distribution tests carried out on five selected samples of the till / residual soil are shown on Figure 3.

Atterberg limit testing was carried out on three samples of the clayey silt till / residual soil deposit, and the results are plotted on a plasticity chart on Figure 4. This limit testing measured plastic limits of 14 to 16 per cent, liquid limits of 22 to 27 per cent, and plasticity indices of 8 to 11 per cent. These results confirm that this portion of the till / residual soil deposit is a clayey silt of low plasticity.

The measured SPT “N” values within the till / residual soil range from 49 to greater than 100 blows per 0.3 m of penetration, but are typically above 75 blows per 0.3 m of penetration. These measured SPT “N” values indicate that the clayey silt portions of this deposit have a hard consistency, while the sandy silt portions of this deposit have a typically very dense relative density.

4.2.6 Shale Bedrock

Red shale bedrock of the Queenston Formation was encountered below the clayey silt till / residual soil in one borehole (Borehole Z-5). At this location, the surface of the shale is at 13.7 m depth (approximately Elevation 119.7 m).

The borehole was advanced into the shale by augering and split-spoon sampling; no limestone or dolostone interbeds were encountered during these operations, although such interbeds are known to be present within the Queenston Formation bedrock. SPT “N” values of greater than 100 blows per 0.3 m of penetration were measured within the shale.

4.3 Groundwater Conditions

All of the boreholes were dry during and upon completion of the drilling operations. A piezometer was installed in Borehole Z-3, screened within the clayey silt till / residual soil, to monitor the groundwater level at the site. The water level in this piezometer was measured at 3.3 m depth (about Elevation 129.9 m) on November 10 and November 22, 2004, approximately one week and three weeks following installation.

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 Foundation Investigation Report was prepared by Ms. Karyn Gallant, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer with Golder. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

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PART B

**FOUNDATION DESIGN REPORT
PLAINS ROAD OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
G.W.P. 19-95-05**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed single-span structure that will carry Zellens Road over the CP Rail line. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the structure foundations. 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. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Bridge and Retaining Wall Foundation Options

At the proposed Zellens Road structure site, the CP Rail line grade is at approximately Elevation 133.5 m, rising eastward. The natural ground surface rises northward, from about Elevation 133 m to 133.5 m south of the rail, to about 134 m to 134.5 m north of the rail. The proposed Zellens Road grade will be at about Elevation 142.5 m at the structure; therefore, the immediate approach embankments will be up to about 9 m in height.

In general, the subsoils at the site consist of about 0.6 m to 0.7 m of firm, surficial clayey silt and stiff clayey silt fill, overlying a very stiff to hard clayey silt to silty clay till deposit; this till grades to a residual soil with depth. Standard Penetration Test (SPT) “N” values of greater than 100 blows per 0.3 m of penetration were measured below about 4 m to 6 m depth (approximately Elevation 128.5 m to 129.5 m) at the proposed south abutment, and below about 4 m to 8 m depth (approximately Elevation 124.5 m to 129.5 m) at the proposed north abutment. Shale bedrock was encountered in one of the boreholes below the till/residual soil, at about Elevation 119.7 m.

The following foundation options may be considered for support of the overhead structure abutments and any associated concrete retaining walls:

- **Spread footings founded on the very stiff to hard glacial till deposit:** This option would be similar to that adopted for the Plains Road and Highway 6 overhead structures on this project.
- **Spread footings “perched” within the approach embankment on a compacted granular pad:** This option has the advantage of minimizing the height of the abutment walls, but would require a longer span bridge than the rigid frame option.

- **Driven steel H-piles, with the pile cap “perched” within the approach embankment fill:** This foundation option could be adopted with either conventional or integral abutments, in conjunction with either 2H:1V abutment foreslopes or with retained soil system (RSS) walls in a “false abutment” configuration.

Recommendations for each of these foundation options are provided in the following sections of this memorandum. From a foundations perspective, “perched” spread footings or driven steel H-pile foundations (with the pile cap perched within the approach embankment) are preferred. However, the use of spread footings supported on the till will afford a shorter structure length and will result in similar structures to those designed for the Highway 6 and Plains Road overhead sites immediately east of the Zellens Road structure site.

6.3 Spread Footings on Glacial Till

The bridge abutments and any associated concrete wing walls / retaining walls may be supported on spread footings founded on the undisturbed, very stiff to hard clayey silt till deposit. As noted in Section 6.3.3, a minimum of 1.2 m of soil cover (or equivalent) must be provided above the footing level to ensure adequate protection against frost penetration.

Requirements for temporary excavation associated with this foundation option, including temporary open-cut excavations, temporary track protection, groundwater control and the protection of the clayey subgrade soils, are addressed in Section 6.9.

6.3.1 Geotechnical Resistance

It is noted that the upper portion of the clayey silt till at the north abutment (as encountered in Boreholes Z-6 and Z-7) is more plastic than that encountered at the south abutment, and so there will be a difference in the factored geotechnical resistance at Ultimate Limit States (ULS) unless the footing in the north abutment is founded deeper. In this regard, the design founding elevations for the south and north abutment footings should be taken at Elevations 132.0 m and 131.0 m, respectively, in order to obtain equivalent factored geotechnical resistances at Ultimate Limit States (ULS). A founding level of Elevation 132.0 m could also be considered for the north abutment; however, a lower factored geotechnical resistance at ULS would apply. The founding elevations and geotechnical resistances can be found in the table in the next section.

The following founding elevations and geotechnical resistances may be used for the design of 4.5 m wide spread footings placed on the properly prepared clayey silt till:

<i>Foundation Element</i>	<i>Founding Elevation</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS*</i>
South abutment	132.0 m	700 kPa	450 kPa
North abutment	132.0 m	550 kPa	400 kPa
	131.0 m	700 kPa	450 kPa

* For 25 mm of settlement.

It is noted that the ULS resistance and settlement are dependent on the footing size, configuration and/or applied loads. The geotechnical resistances should, therefore, be reviewed 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 cohesive soils.

6.3.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, between cast-in-place concrete footings and the properly prepared, very stiff to hard clayey silt till 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.

6.3.3 Frost Protection

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

6.4 Spread Footings Perched in Approach Embankment

In order to minimize the height of the abutment walls, spread footings for the abutments may be placed on a compacted Granular “A” pad within the approach embankment fill.

6.4.1 Geotechnical Resistance

Assuming that the compacted Granular “A” pad has a thickness of at least one footing width, the footing design may be carried out using a factored geotechnical resistance at ULS of 900 kPa, and a geotechnical resistance at SLS of 350 kPa. These geotechnical resistances 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.

6.4.2 Resistance to Lateral Loads

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

6.5 Driven Steel H-Pile Foundations

The abutments may be supported on steel H-piles driven to found within the 100-blow glacial till / residual soil; the surface of the 100-blow soil was encountered in the boreholes between about Elevations 124.5 m and 129.5 m. Driving shoes will be required for protection during pile installation, due to the hard / very dense nature of the till / residual soil, and the potential presence of cobbles and boulders within this glacially-derived material. For design, it may be assumed that the piles will be founded at about Elevation 125 m at the south abutment, and about Elevation 124 m at the north abutment. It is assumed that the pile caps would be perched within the approach embankments, with the underside of the pile cap at or above approximately Elevation 139 m. The piles would therefore be approximately 15 m in length.

6.5.1 Axial Geotechnical Resistance

The factored axial resistance at ULS for steel HP 310 x 110 piles driven to found within the till / residual soil may be taken as 1,600 kN. The settlement of the individual piles and the pile group at this pile load is anticipated to be less than 25 mm. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and, as such, the ULS conditions will govern at this site. The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. The criteria must therefore be established at the time of construction after the piling equipment is known. The following note is considered appropriate for the design and site conditions, assuming a resistance factor of 0.4 is applied to the use of the Hiley formula:

Piles to be driven in accordance with Standard SS 103-11, using an ultimate capacity of 4,000 kN per pile.

6.5.2 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. If integral abutments are under consideration for this structure, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the equations given below.

For cohesive soils:

$$k_h = \frac{k_{s1}}{5B} \quad \text{where} \quad \begin{array}{l} k_{s1} \text{ is the coefficient of horizontal subgrade reaction; and} \\ B \text{ is the pile diameter (m).} \end{array}$$

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where} \quad \begin{array}{l} n_h \text{ is the constant of horizontal subgrade reaction;} \\ z \text{ is the depth (m); and} \\ B \text{ is the pile diameter (m).} \end{array}$$

The following ranges for the value of k_{s1} and n_h may be assumed in the structural analysis. The range in values reflects the variability in the subsurface conditions as well as the two extremes of design: the requirement for flexibility in the case of integral abutments, and the requirement for lateral support in the case of non-integral abutments.

<i>Soil Unit</i>	<i>k_{s1}</i>	<i>n_h</i>
New embankment fill (assumed to be compacted granular fill)	—	5 to 15 MPa/m
South abutment:		
Stiff clayey silt fill, above approximately Elevation 133 m	15 to 25 MPa/m	—
Very stiff to hard clayey silt till, above Elevation 130 m	45 to 70 MPa/m	—
Hard till/residual soil, between Elevations 130 m and 126 m	70 to 150 MPa/m	—
Very dense till/residual soil, between Elevations 126 m and 122 m	—	20 to 30 MPa/m
Hard till/residual soil, below Elevation 122 m	100 to 150 MPa/m	—
North abutment:		
Stiff clayey silt fill, above approximately Elevation 133 m	15 to 25 MPa/m	—
Very stiff to hard clayey silt till, above about Elevation 129 m	45 to 70 MPa/m	—
Hard clayey silt till/residual soil, below about Elevation 129 m	70 to 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

6.5.3 Frost Protection

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

6.6 Retained Soil System (RSS) Walls

Since the anticipated long-term settlements beneath retaining walls associated with the Zellens Road overhead structure are estimated to be less than 25 mm based on the proposed embankment loading, the use of a mechanically-reinforced soil retaining wall system (RSS wall) is considered suitable for this site. RSS walls located adjacent to the abutment walls at this site are expected to be between 5 m and 7 m high.

The reinforced earth mass should be placed after stripping the topsoil and any loose fill / surficial deposits. Based on the results of the boreholes, the RSS walls will be founded on the thin layer of firm to stiff surficial clayey silt or clayey silt fill (where present) or on the stiff to hard clayey silt till. Assuming that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass, which is taken as 70 per cent of the height of the wall at any given location, the factored geotechnical resistances at ULS and the geotechnical resistance at SLS (for 25 mm of settlement) given in the following table may be used for assessment of the reinforced mass founded on the properly prepared subgrade.

<i>Wall Height</i>	<i>Assumed Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
5 m	3.5 m	200 kPa	200 kPa
7 m	4.9 m	300 kPa	200 kPa

The facing panels must be placed on a granular levelling pad; based on the subsurface conditions encountered at this site, the thickness of the levelling pad may be taken as 300 mm. However, provision should be made in the contract for the subexcavation of soft or loose fill materials if these are encountered at subgrade level, prior to placement of the granular levelling pad.

The resistance to lateral forces / sliding resistance between the compacted granular fill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fill of the RSS wall and the existing granular fill materials or the native sand to sand and gravel soils may be taken as 0.62. 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.

6.7 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(I) 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(I) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the existing and proposed embankment fill materials and the following parameters (unfactored) may be assumed:

Soil unit weight:	20 kN/m ³
Coefficients of lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

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

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

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

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.

6.8 Approach Embankments

The construction of the Zellens Road approach embankments will require placement of up to 9 m of fill above the existing ground surface behind the proposed abutments. Based on the borehole results, the embankment subgrade soils will consist of a thin layer of existing clayey silt fill or surficial clayey silt (where present) overlying a generally very stiff to hard clayey silt till deposit.

6.8.1 Subgrade Preparation and Embankment Construction

Any topsoil, organic matter and softened/loosened soils should be stripped from below the approach embankment areas, and all subgrade soils proof-rolled prior to placement of fill for the approach embankments. The embankment fill should be placed and compacted in accordance with OPSS 501. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

6.8.2 Embankment Stability

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

The static slope stability analyses were carried out using the following parameters, based on field and laboratory test data and accepted correlations:

<i>Soil</i>	<i>Bulk Unit Weight</i>	<i>Effective Friction Angle</i>	<i>Undrained Shear Strength</i>
Embankment fill (range of parameters assuming earth and granular fill)	20 – 22 kN/m ³	30° – 35°	–
Clayey silt fill / Surficial clayey silt	20 kN/m ³	28°	50 kPa
Very stiff to hard clayey silt till	21 kN/m ³	32° - 35°	–

The results of the slope stability analyses indicate that the 9 m high approach embankments with side slopes oriented at 2 horizontal to 1 vertical (2H:1V) will have a minimum factor of safety of 1.3 against deep-seated slope instability, assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials. A mid-height berm will be required where the approach embankment height is equal to or greater than 8 m.

6.8.3 Embankment Settlement

Settlement of the approach embankments will occur as a result of compression of the new embankment fill itself, as well as consolidation of the clayey soils on which the approaches will be founded.

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

Settlement analyses for the foundation soils were carried out using the commercially-available computer program Unisettle. The compression of the surficial clayey silt / clayey silt fill and the clayey silt till deposits was modelled using elastic deformation moduli based on correlations with the measured SPT “N” values and preconsolidation pressures, assuming the till material to be overconsolidated. The parameters used in the analyses are summarized in the following table:

<i>Soil</i>	<i>Bulk Unit Weight</i>	<i>Elastic Modulus</i>
Embankment fill (range of parameters assumed for earth fill and granular fill)	20 – 22 kN/m ³	–
Firm to stiff clayey silt fill and surficial clayey silt	20 kN/m ³	15 MPa
Very stiff to hard clayey silt till	21 kN/m ³	50 MPa

Based on the above, the settlement of the foundation soils below the 9 m high approach embankment loading is calculated to be approximately 20 mm to 25 mm. The majority of this compression is expected to occur within about three to six months of completion of the embankment construction.

6.9 Construction Considerations

6.9.1 Temporary Open-Cut Excavations

If spread footings supported on the clayey silt till deposit are adopted for the Zellens Road overhead structure, excavations for construction of the abutment footings would extend to about 1.5 m to 2.5 m below the existing rail grade, based on the design founding elevations given in Section 6.3. This will require excavation through the existing fill and surficial clayey silt, where present, and into the generally very stiff to hard clayey silt till. Temporary open-cut excavations should be carried out in accordance with the guidelines outlined in the Occupation Health and Safety Act and Regulations for Construction Activities. The site soils would be classified as Type 2 or 3, according to the OHSA, and therefore temporary excavations should be formed with side slopes no steeper than 1 horizontal to 1 vertical.

6.9.2 Temporary Track Protection

Where space restrictions preclude the use of temporary open-cuts, as is likely to be the case where spread footing excavations are advanced to between 1.5 m and 2.5 m depth adjacent to the CP Rail line, temporary track protection will be required. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision 539S01. The lateral movement of the temporary shoring system at this location should meet Performance Level 2 as specified in SP 539S01.

6.9.3 Groundwater and Surface Water Control

Excavations for spread footings would extend to between 1.5 m and 2.5 m depth, and will be maintained above the groundwater level at the site; however, some seepage into the excavations should be expected from groundwater "perched" within any granular fill on top of the clayey silt till deposit. It is considered that the quantity of groundwater seepage can be handled by pumping from properly filtered sumps placed at the base of the excavation. The sumps should be maintained outside the footing limits.

It should be noted that the soils in which the footing excavations will be formed are susceptible to disturbance from ponded water and construction traffic. Provision should be made in the Contract Documents for the placement of a lean concrete mat to protect the soils from such disturbance. Such a working mat should be placed within four hours after subgrade preparation and inspection.

6.9.4 Obstructions

The native soils at the site are glacially-derived and, as such, are expected to contain cobbles and boulders. The presence of such obstructions will affect the installation of driven steel H-piles for deep foundations or temporary excavation support, and will also affect the installation of soldier piles and soil or rock anchors (tie-backs) if these are necessary for the temporary excavation support system. Provision should be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Karyn Gallant, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and Senior Engineer, with input from Ms. Anne Poschmann, a Principal and Senior Engineer with Golder. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

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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

(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 $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

PROJECT 001-1141F			RECORD OF BOREHOLE No BH Z-3			1 OF 1			METRIC								
W.P. 19-95-00			LOCATION N 4795210.7 ; E 272172.3			ORIGINATED BY PKS											
DIST Central HWY 6			BOREHOLE TYPE 108 mm Diameter Solid Stem Augers			COMPILED BY KG											
DATUM Geodetic			DATE November 2, 2004			CHECKED BY LCC											
SOIL PROFILE			SAMPLES			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	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa		WATER CONTENT (%)		γ	GR SA SI CL				
							20 40 60 80 100	○ UNCONFINED + FIELD VANE	20 40 60 80 100	W _p W W _L	10 20 30						
133.2	GROUND SURFACE																
0.0	Topsoil		1	SS	14		132										
0.2	Clayey Silt, trace to some sand, trace gravel, shale and limestone fragments (TILL) Stiff to hard Brown to red-brown Moist		2	SS	48		132										
			3	SS	42		131										2 8 52 38
			4	SS	40		130										
			5	SS	37		129										
129.1	Clayey Silt, trace to some sand, trace gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Hard Red-brown/grey Moist		6	SS	37		128										
4.1			7	SS	74		127										
127.6	Clayey Silt, trace sand and gravel (TILL/RESIDUAL SOIL) Hard Grey Moist		8	SS	51												
5.6																	
126.5	End of Borehole																
6.7	Notes: 1. Open borehole dry upon completion of drilling. 2. Water level in piezometer measured at 3.3 m depth (Elev. 129.9 m) on November 10, and November 22, 2004.																

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PROJECT 001-1141F		RECORD OF BOREHOLE No BH Z-4		1 OF 1	METRIC
W.P. 19-95-00		LOCATION N 4795214.6; E 272149.7		ORIGINATED BY PKS	
DIST Central HWY 6		BOREHOLE TYPE 108 mm Diameter Solid Stem Augers		COMPILED BY KG	
DATUM Geodetic		DATE November 3, 2004		CHECKED BY LCC	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
134.4	GROUND SURFACE										
133.7	Topsoil Clayey Silt, some sand, trace gravel and shale fragments (FILL) Stiff Brown Moist		1	SS	12						
133.7	Clayey Silt, some sand, trace gravel, shale and limestone fragments (TILL) Very stiff to hard Brown to red-brown Moist		2	SS	54						
			3	SS	74						
			4	SS	48						
			5	SS	48						
			6	SS	28						
129.9	Clayey Silt, trace sand and gravel (TILL/RESIDUAL SOIL) Hard Grey Moist		7	SS	61						
128.7	Sandy Silt, trace clay, gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Very dense Red-brown Moist to wet		8	SS	127						
127.4	Clayey Silt, trace sand and gravel, containing silt seams (TILL/RESIDUAL SOIL) Hard Grey Moist		9	SS	55						
125.7	Sandy Silt, trace clay, trace to some gravel, trace shale and limestone fragments (TILL/RESIDUAL SOIL) Very dense Red-brown Dry to moist		10	SS	100/20						
				SS	100/100						
121.9	End of Borehole		12	SS	100/10						
121.9	Notes: Open borehole dry upon completion of drilling.										

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RECORD OF BOREHOLE No BH Z-5

1 OF 2

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4795237.4, E 272165.0

ORIGINATED BY PKS

DIST Central HWY 6

BOREHOLE TYPE 108 mm Diameter Solid Stem Augers

COMPILED BY KG

DATUM Geodetic

DATE November 3, 2004

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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		SHEAR STRENGTH kPa						
133.4	GROUND SURFACE						20 40 60 80 100		10 20 30				
0.0	Topsoil												
0.2	Clayey Silt, some sand, trace gravel and organicsl (FILL) Stiff		1	SS	12								
132.5	Brown												
0.9	Moist		2	SS	39								
	Clayey Silt, some sand, trace gravel, shale and limestone fragments (TILL) Hard		3	SS	49								
	Brown to red-brown												
	Moist		4	SS	47								
			5	SS	49								
129.7													
3.7	Clayey Silt, some sand, trace gravel, shale and limestone pieces (TILL/RESIDUAL SOIL) Hard		6	SS	125								
	Red-brown												
	Moist		7	SS	122								10 15 57 18
127.8													
5.6	Clayey Silt, trace sand and gravel, containing silt seams (TILL/RESIDUAL SOIL) Hard		8	SS	49								
	Grey												
	Moist												
126.3													
7.2	Sandy Silt, some clay and gravel, trace shale and limestone fragments (TILL/RESIDUAL SOIL) Very dense		9	SS	100/ 25								17 19 53 11
	Red-brown												
	Moist		10	SS	100/ 18								
123.4													
10.1	Clayey Silt, some sand, trace gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Hard		11	SS	75								
	Red-brown												
	Moist		12	SS	100/ 18								
119.7													
13.7	Shale (BEDROCK) Red		13	SS	100/ 0.2								
		</											

Continued Next Page

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

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RECORD OF BOREHOLE No BH Z-5

2 OF 2

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4795237.4 ; E 272165.0

ORIGINATED BY PKS

DIST Central HWY 6

BOREHOLE TYPE 108 mm Diameter Solid Stem Augers

COMPILED BY KG

DATUM Geodetic

DATE November 3, 2004

CHECKED BY LCC

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			W _p
118.2	--- CONTINUED FROM PREVIOUS PAGE ---														
15.3	End of Borehole Notes: Open borehole dry upon completion of drilling.		11	SS	100.0		110								

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PROJECT <u>001-1141F</u>		RECORD OF BOREHOLE No BH Z-6		1 OF 1	METRIC
W.P. <u>19-95-00</u>		LOCATION <u>N 4795223.1 ; E 272141.1</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>6</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>KG</u>	
DATUM <u>Geodetic</u>		DATE <u>November 5, 2004</u>		CHECKED BY <u>LCC</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE 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	20 40 60 80 100	10 20 30		
133.5	GROUND SURFACE													
0.0	Topsoil													
0.2	Clayey Silt, some sand, trace gravel Firm		1	SS	8									
132.8	Brown to dark brown Moist													
0.7	Clayey Silt to Silty Clay, some sand, trace gravel, shale and limestone fragments (TILL) Very stiff to hard		2	SS	39									
	Brown to red-brown Moist													
			3	SS	39									
			4	SS	27									
			5	SS	31									
			6	SS	42									
			7	SS	47									
127.9	Clayey Silt, trace sand and gravel (TILL/RESIDUAL SOIL) Hard Grey Moist		8	SS	58									
126.5	Clayey Silt, some sand, trace gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Hard Red-brown Moist		9	SS	64									
125.1	Sandy Silt, trace clay, trace gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Very dense Red-brown Moist to wet		10	SS	100/23									17 17 52 14
121.2			11	SS	100/23									
121.2	End of Borehole		12	SS	100/15									
12.3	Notes: No standing water upon completion of drilling.													

MISS MTO 0011141FAAMTO.GPJ ON MOT.GDT 19/4/05

RECORD OF BOREHOLE No BH Z-7

1 OF 1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4795246.1 E 272156.3

ORIGINATED BY PKS

DIST Central HWY 6

BOREHOLE TYPE 108 mm Diameter Solid Stem Augers

COMPILED BY KG

DATUM Geodetic

DATE November 8, 2004

CHECKED BY LCC

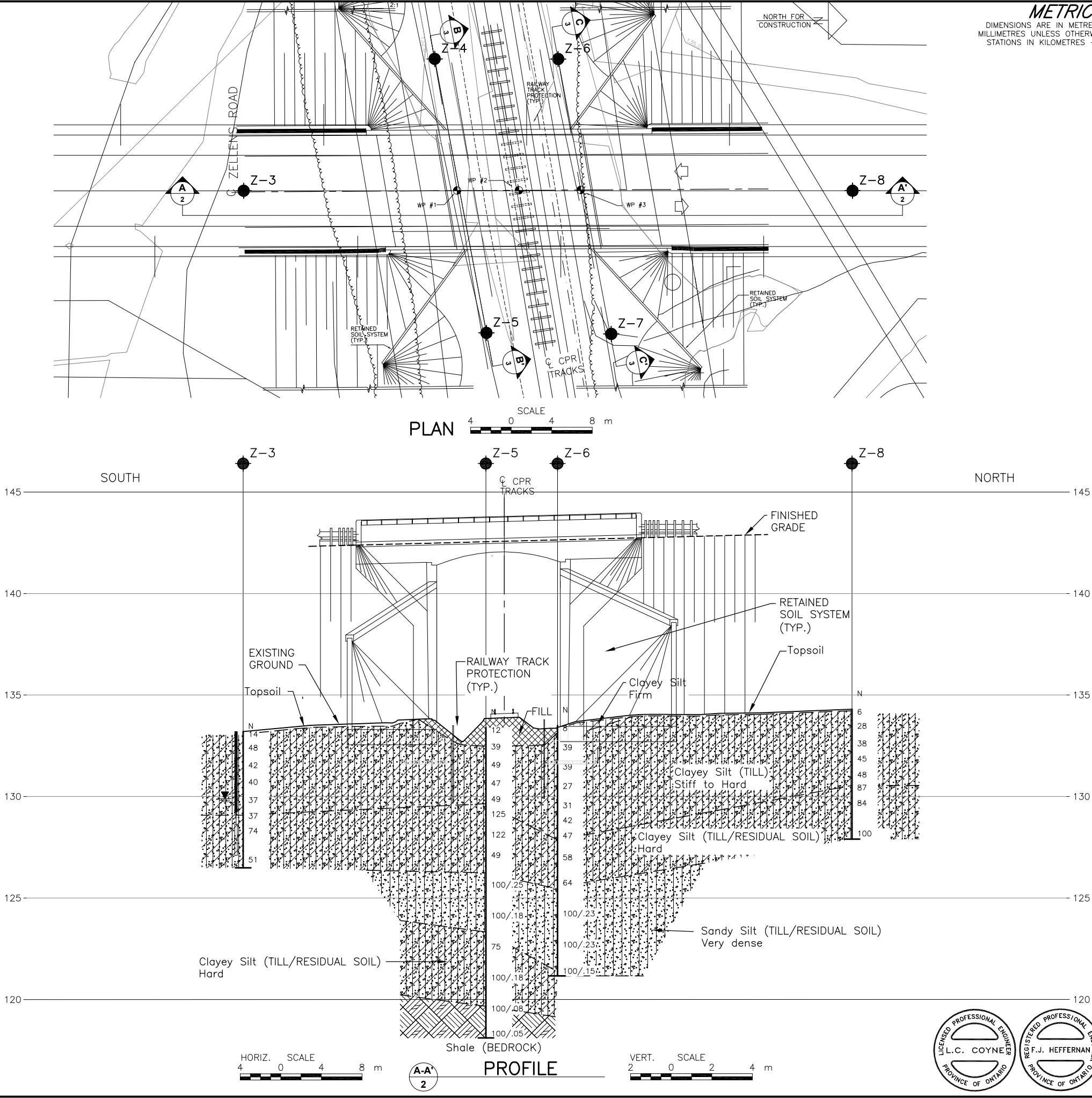
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
133.5	GROUND SURFACE							20 40 60 80 100				
0.0	Topsoil							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X REMOULDED				
0.3	Clayey Silt to Silty Clay, some sand, trace gravel, shale and limestone fragments (TILL) Very stiff to hard Red-brown Moist		1	SS	9		133	WATER CONTENT (%) W _p W W _L				
			2	SS	23		132					
			3	SS	36		131					
			4	SS	52		130					
			5	SS	49		129					
129.8	Clayey Silt, trace to some sand, trace to some gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Hard Red-grey to red-brown Moist		6	SS	50/05		128					
3.7			7	SS	102		127					
			8	SS	50		126					
			9	SS	100/23		125					11 21 52 16
			10	SS	100/20		124					
							123					
122.4	End of Borehole		11	SS	150							
11.1	Notes: Open borehole dry upon completion of drilling.											

MISS MTO 0011141FAAMTO GPJ ON MOT.GDT *18/4/05

PROJECT <u>001-1141F</u>		RECORD OF BOREHOLE No BH Z-8		1 OF 1	METRIC
W.P. <u>19-95-00</u>		LOCATION <u>N 4795252.7 :E 272129.4</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>6</u>		BOREHOLE TYPE <u>108 mm Diameter Solid Stem Augers</u>		COMPILED BY <u>KG</u>	
DATUM <u>Geodetic</u>		DATE <u>November 5, 2004</u>		CHECKED BY <u>LCC</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 C
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								\circ UNCONFINED \bullet QUICK TRIAXIAL	$+$ FIELD VANE \times REMOULDED	w_p	w	w_L		
134.3	GROUND SURFACE						20 40 60 80 100							
0.0	Topsoil													
0.2	Clayey Silt, some sand, trace gravel, shale and limestone fragments (TILL) Firm to hard Red-brown Moist		1	SS	6		134							
			2	SS	28		133							
			3	SS	38		132							
			4	SS	45		131							
			5	SS	48		130							
130.6	Clayey Silt, trace sand, gravel, shale and limestone fragments (TILL/RESIDUAL SOIL) Hard Red-grey Moist		6	SS	87		130							
3.7			7	SS	84		129							
							128							
127.9	End of Borehole		8	SS	100		127							
6.4	Notes: Open borehole dry upon completion of drilling.													

MISS MTO 0011141FAAMTO.GPJ ON MOT.GDT 18/4/05



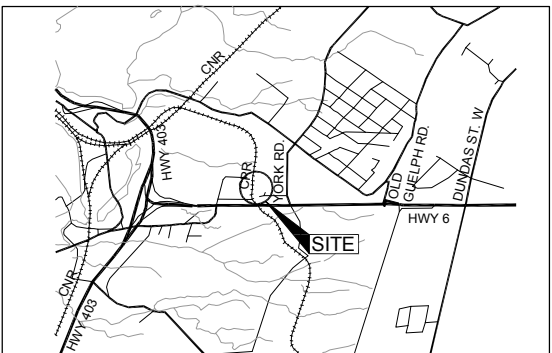
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 19-95-05

ZELLENS ROAD OVERHEAD
BOREHOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
Z-3	133.2	4795210.7	272172.3
Z-4	134.4	4795214.6	272149.7
Z-5	133.4	4795237.4	272165.0
Z-6	133.5	4795223.1	272141.1
Z-7	133.5	4795246.1	272156.3
Z-8	134.3	4795252.7	272129.4

NOTES

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

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

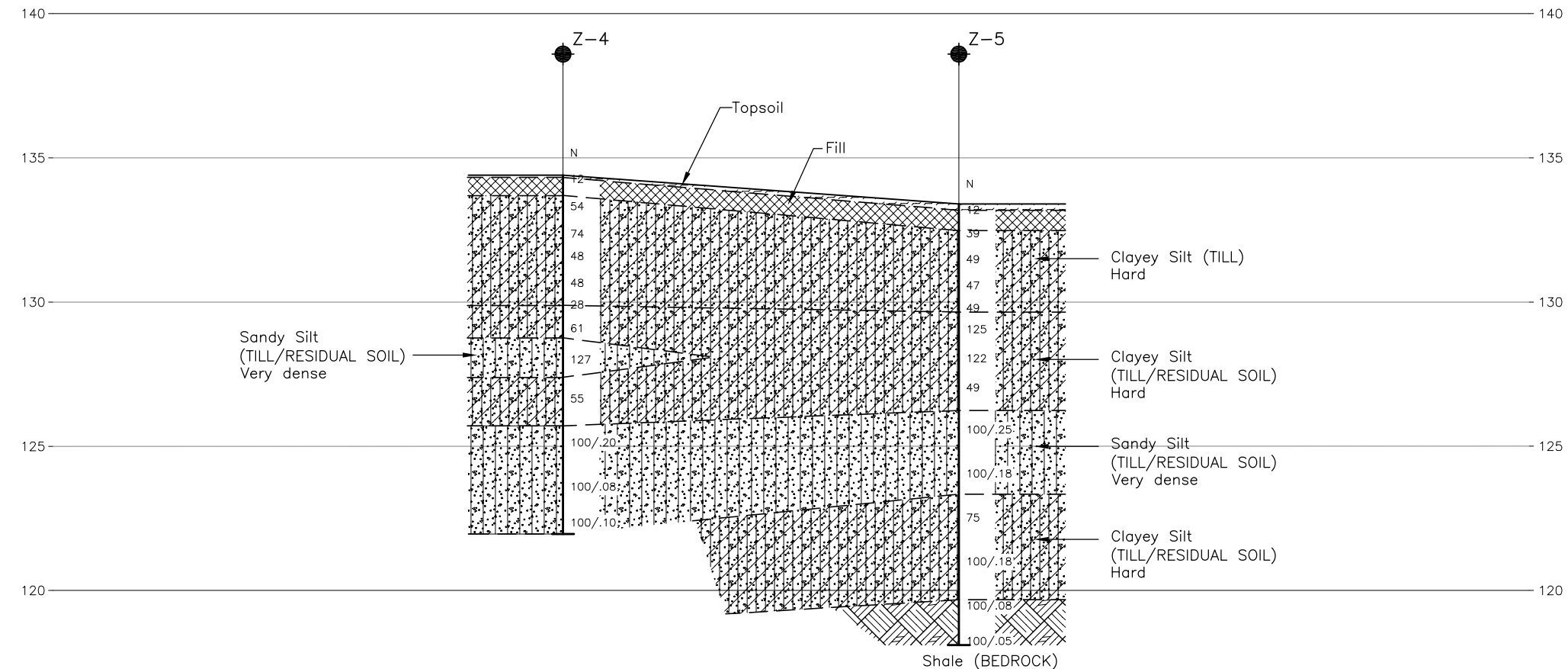
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

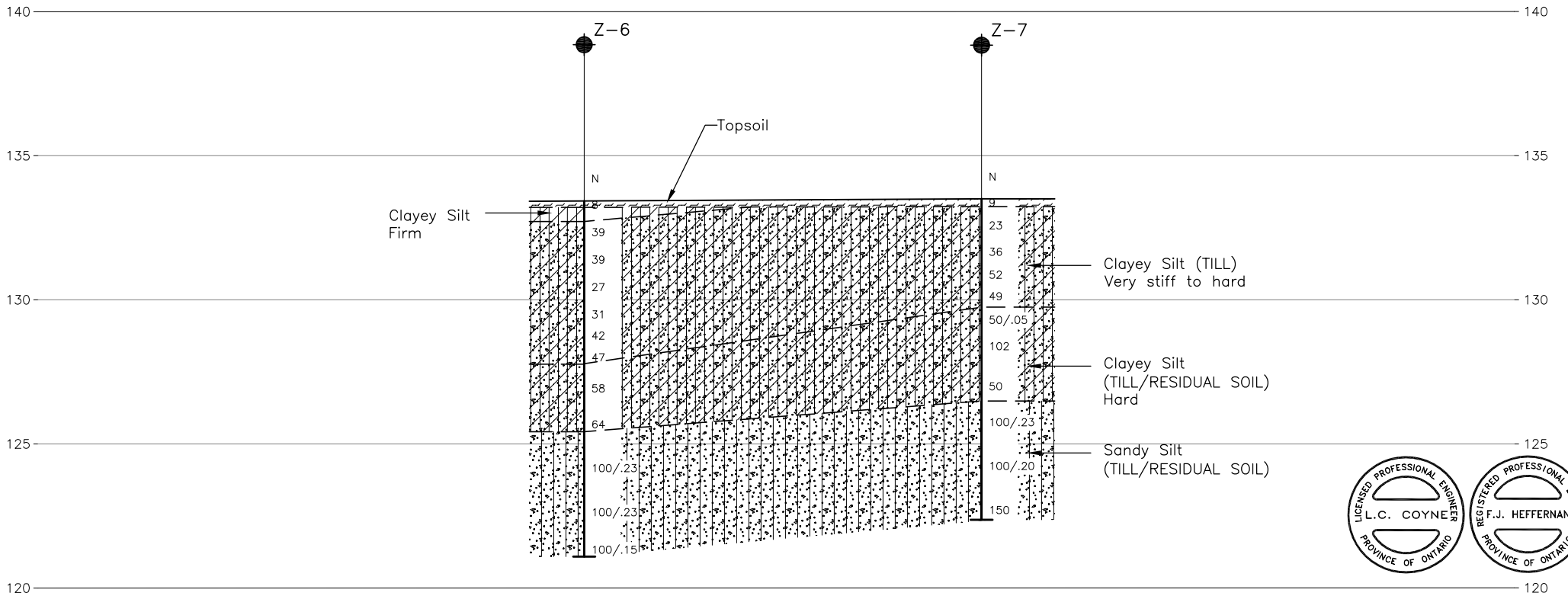
General Arrangement file (ZellensGA.dwg) provided in digital format by URS Canada Inc., received April 1, 2005.

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 6	PROJECT NO. 001-1141F		DIST.
SUBM'D. PKS	CHKD. LCC	DATE: MARCH 2005	SITE:
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1

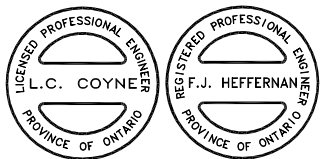
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.



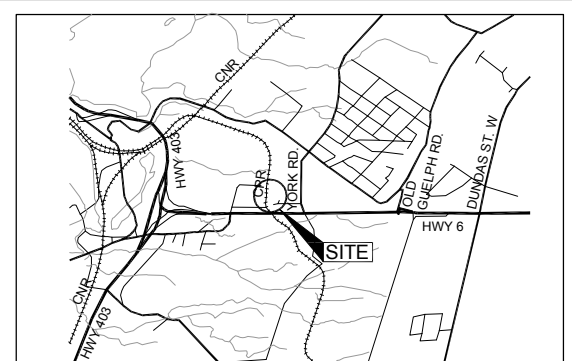
B-B' 2 CROSS SECTION ALONG SOUTH ABUTMENT



C-C' 2 CROSS SECTION ALONG NORTH ABUTMENT



CONT No. WP No. 19-95-05	
ZELLENS ROAD OVERHEAD SOIL STRATA	SHEET



KEY PLAN

LEGEND			
	Borehole - Current Investigation		
	Seal		
	Piezometer		
	Standard Penetration Test Value		
	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)		
	WL in piezometer		
	WL upon completion of drilling		

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
Z-4	134.4	4795214.6	272149.7
Z-5	133.4	4795237.4	272165.0
Z-6	133.5	4795223.1	272141.1
Z-7	133.5	4795246.1	272156.3

NOTES

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

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

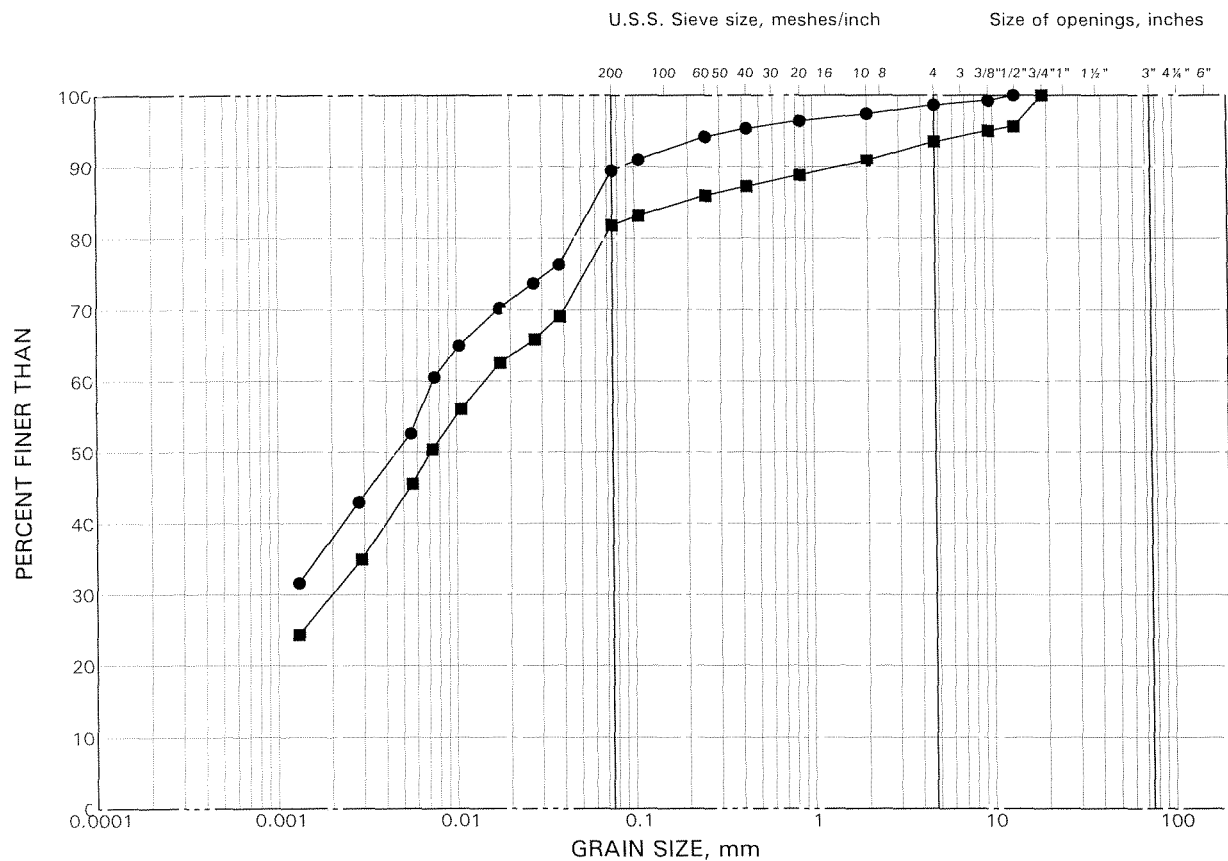
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NO.	DATE	BY	REVISION
Geocres No.			
HWY. 6	PROJECT NO. 001-1141F		DIST.
SUBM'D. PKS	CHKD. LCC	DATE: MARCH 2005	SITE:
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt to Silty Clay Till

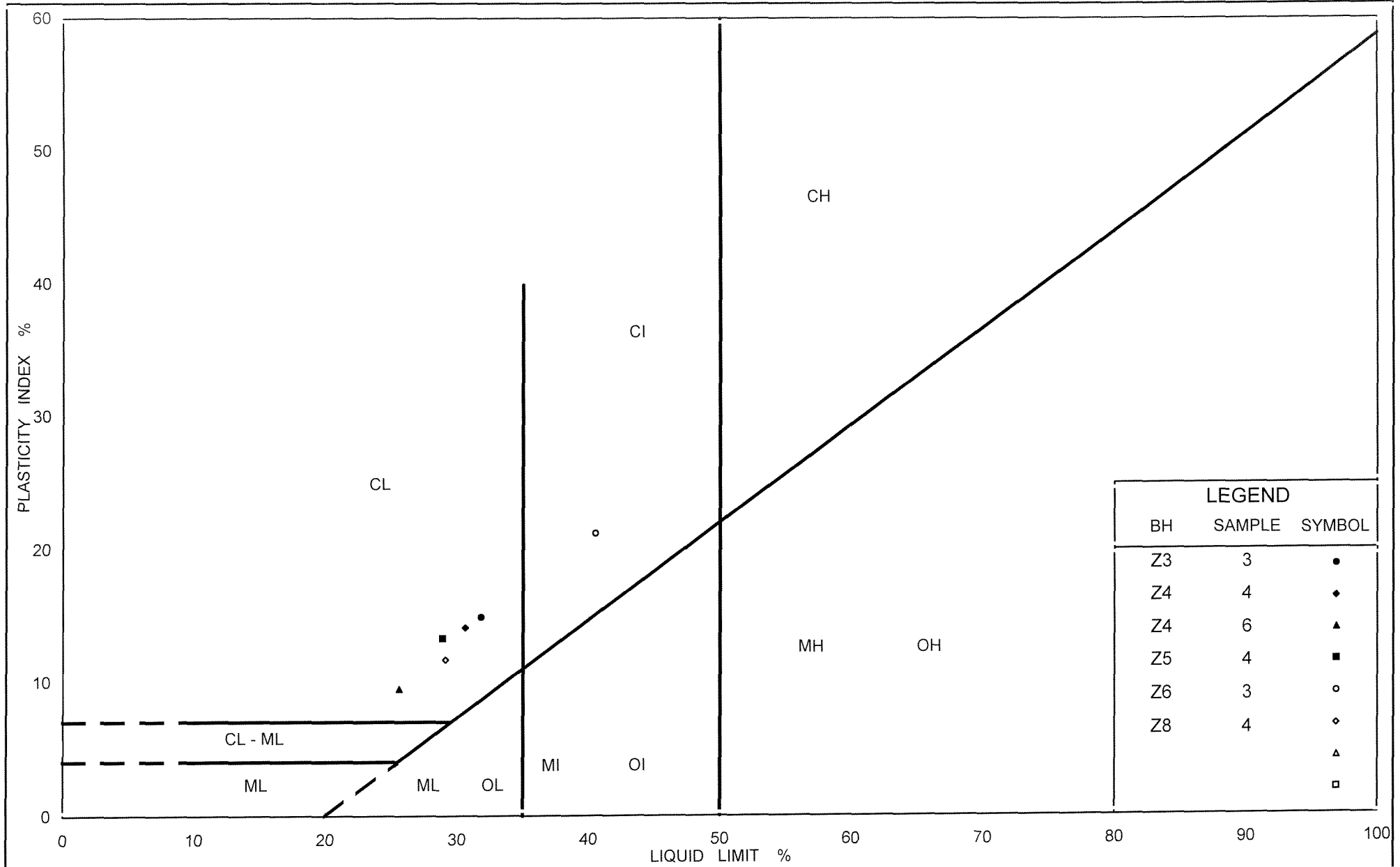
FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	Z3	3	131.3
■	Z4	4	131.8



Ontario

Ministry of Transportation

PLASTICITY CHART Clayey Silt to Silty Clay Till

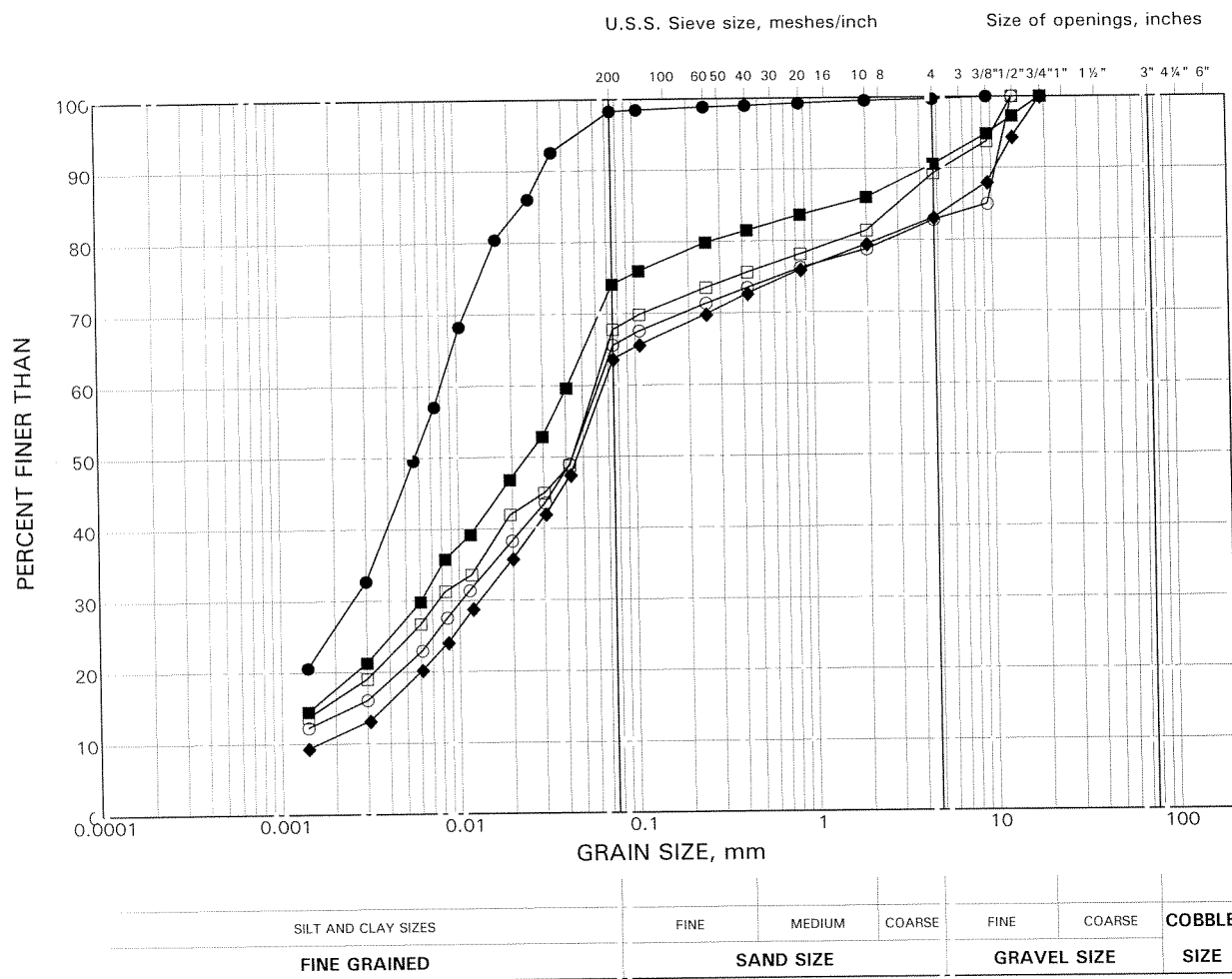
FIG No.2

Project No. 001-1141F

GRAIN SIZE DISTRIBUTION TEST RESULTS

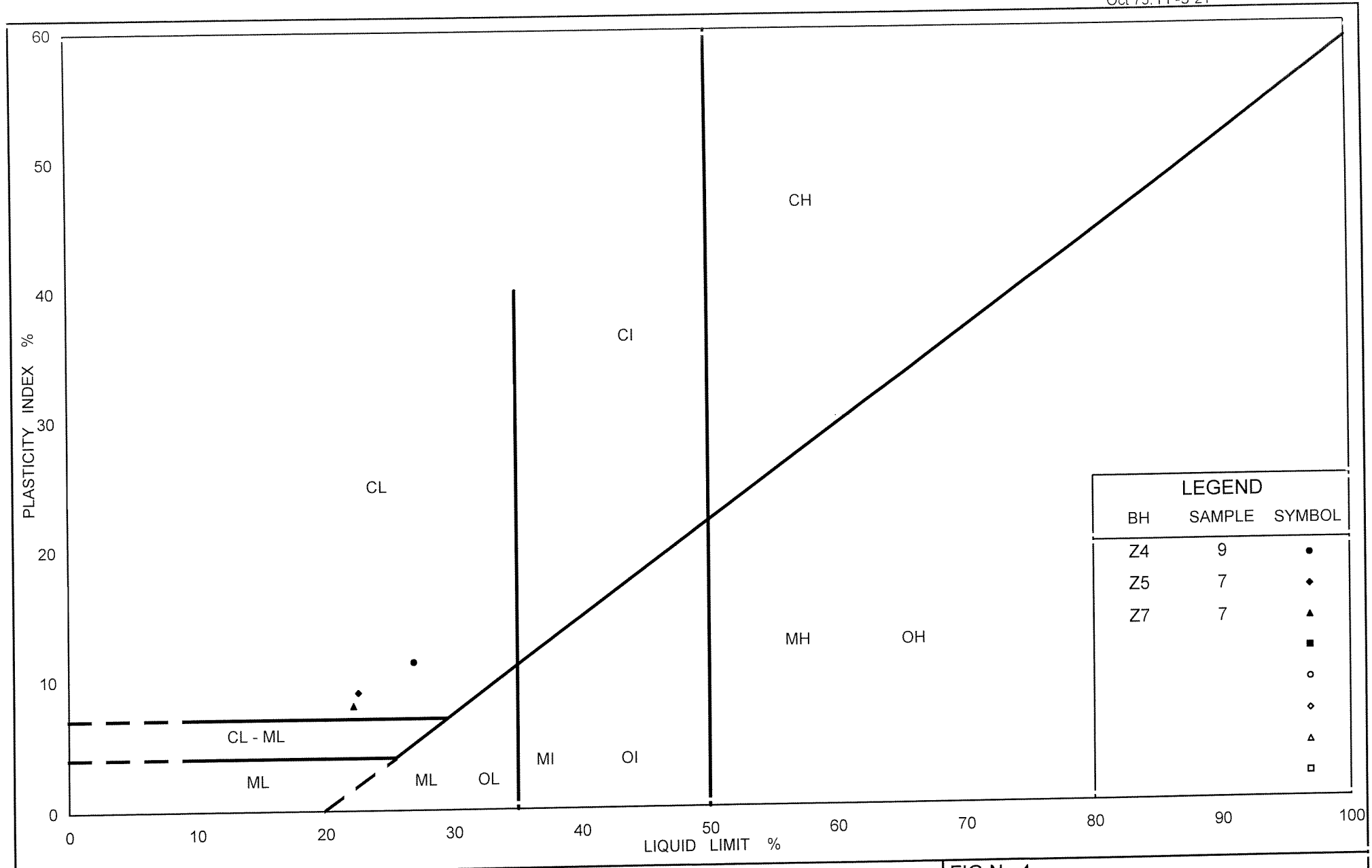
Clayey Silt to Sandy Silt Till / Residual Soil

FIGURE 3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	Z4	9	126.4
■	Z5	7	128.6
◆	Z5	9	125.7
○	Z6	10	124.2
□	Z7	9	125.8



PLASTICITY CHART
Clayey Silt Till / Residual Soil

FIG No.4

Project No. 001-1141F



Ministry of Transportation

Ontario