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FOUNDATION
INVESTIGATION AND DESIGN REPORT
HIGHWAY 6 OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING
BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-04

Submitted to:

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November 2002 001-1141F-1

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PART A

FOUNDATION INVESTIGATION REPORT
HIGHWAY 6 OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-04

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 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 replacement of the existing three-span Highway 6 bridge over the CP Rail alignment. A foundation investigation has been carried out to determine the subsurface conditions at the site. Subsurface data from a 1960 investigation carried out for the Department of Highways, Ontario by Dominion Soil Investigation Ltd. *(Report on Foundation Investigation, Proposed CPR Overhead on Highway 6, 1.2 Miles South of Clappison's Corners, District No. 4, W.P. 287-60,* dated November 1960) were used to supplement the data obtained in the current investigation.

The terms of reference for the scope of work are outlined in Golder's Proposal No. P01-1166, dated June 2000. The work has been carried out in accordance with Golder's 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 existing Highway 6 – CP Rail structure, the natural ground surface varies from about Elevation 145 m to 140 m; the ground surface generally declines toward the south and west of the structure. Highway 6 has been constructed on an embankment up to about 6 m high, with the existing highway grade at about Elevation 146.5 m to 147 m at the structure. The CP Rail line has been constructed in a cut between 2 m and 6 m deep, with the rail grade at about Elevation 138.8 m to 138 m below Highway 6. The rail grade and the cut depth decline from east to west at the structure site.

The existing three-span bridge carrying Highway 6 over CP Rail was constructed in the early 1960s, to replace the then-existing two-lane, three span bridge. According to the General Layout plan for the existing structure *(General Layout, Clappison's Corners CPR Overhead 2 Miles South of Highway 5, Site 36-20, W.P. 287-60,* dated November 1963), the structure is supported on spread footings, with the north abutment founded at about Elevation 142 m, the piers founded at about Elevation 136.6 m, and the south abutment founded at about Elevation 140.5 m. The existing spread footings are about 3.2 m to 3.5 m wide according to the General Layout drawing.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the Highway 6 overpass at CP Rail in January 2001 and October 2002, at which time seven boreholes were advanced. The locations at which boreholes could be drilled were restricted due to the relatively steep highway embankment side slopes, existing abutment foreslopes, clearance distances from the CP Rail line, and the presence of a fibre optic cable along the north side of the rail line. Use was also made of three boreholes advanced during a 1960 subsurface investigation program at this site by Dominion Soil Investigation Ltd. on behalf of the Department of Highways, Ontario (DHO).

Five of the boreholes advanced in January 2001 (H3, H4, P1, P4 and P5) were drilled to depths of about 11 m to 17 m, to extend through the existing fill (where present) and into the underlying hard clayey silt till. These boreholes were advanced by solid stem augers using a bombardier-mounted drill rig supplied and operated by Master Soil Investigations Ltd. of Weston, 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 H3 to monitor the groundwater level at the site.

Two of the boreholes (H1 and H2) were advanced for the north and south approach embankments on the west side of the highway. Due to access restrictions, these approach embankment boreholes were advanced by portable hand-held and tripod-mounted equipment, supplied and operated by Sonic Soil Sampling Ltd. of Concord, Ontario. Continuous samples of the overburden were obtained using a 50 mm outside diameter split-spoon-type sampler, 0.75 m in length, which was advanced with a half-weight hammer. These boreholes were advanced to about 3 m and 5.5 m depth respectively, to extend through the embankment fill into the underling clayey silt till.

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 Limits testing and grain size distribution analyses were carried out on selected soil samples.

The borehole locations and ground surface elevations were measured by Callon Dietz, Ontario Land Surveyors or were determined by Golder relative to points staked by Callon Dietz on the foundation elements. The borehole locations, including MTM NAD27 northing and easting coordinates, and ground surface elevations referenced to geodetic datum are shown on Drawing 1.

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 Hamilton area, 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

As part of the subsurface investigation for this structure site, seven boreholes were advanced in 2001 and 2002; use was also made of three boreholes advanced in 1960 by the Department of Highways, Ontario for the 1960s construction of the existing three-span structure. 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 4. 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.

In summary, the soils encountered at this site consist of up to about 5.5 m of clayey silt fill associated with the existing Highway 6 embankment slopes and the existing abutment foreslopes. The fill overlies a deposit of hard, brown to grey-brown clayey silt till, which in turn overlies a deposit of hard, red-brown clayey silt till / residual soil. In one of the boreholes, the till / residual soil deposit was penetrated and found to be underlain by red-brown shale bedrock. The surface of the shale was encountered in this borehole at Elevation 129.7 m (about 8 m to 9 m below the rail cut grade, and 16 m to 17 m below the existing highway grade). A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Embankment and Abutment Foreslope Fill

Boreholes H1 to H4 and 60-3 encountered between 1.4 m and 5.5 m of fill associated with the existing Highway 6 embankment and the north and south abutment foreslopes. The fill consists of brown clayey silt containing trace sand and gravel; black cinders or slag were encountered within the fill in two of the boreholes.

The existing fill is generally very stiff to hard, based on measured Standard Penetration Test (SPT) 'N' values ranging from 15 to 35 blows per 0.3 m of penetration. The upper 1 m of embankment fill encountered in Borehole H2, at the south approach on the west side of the existing Highway 6, was stiff based on measured SPT 'N' values of 7 and 9 blows per 0.3 m of penetration.

4.2.2 Clayey Silt to Silty Clay Till

The embankment and abutment foreslope fill is underlain by a deposit of brown to grey-brown clayey silt till, which grades to a silty clay till at some of the borehole locations. This till contains trace to some sand, and trace gravel and shale fragments. Grain size distribution test results obtained on two samples of this till are shown on Figure 1 following the text of this report. The base of this till deposit was encountered between about Elevation 133 m and 130 m, approximately 5 m to 8 m below the existing rail cut grade.

Atterberg limits testing carried out on eight samples of this till measured plastic limits of 15 to 17 per cent and liquid limits of 27 to 37 per cent, with corresponding plasticity indices of 12 to 19 per cent. The results of the limits testing, shown on Figure 2, indicate that the till is predominantly an inorganic clayey silt of low plasticity, although portions of the till grade to an intermediate plasticity silty clay. The measured natural moisture contents range from 8 to 18 per cent, typically at or slightly below the plastic limit for the material.

The till has a hard consistency, with measured SPT “N” values ranging from 37 to greater than 100 blows per 0.3 m of penetration.

4.2.3 Clayey Silt Till / Residual Soil

The brown to grey-brown clayey silt to silty clay till is underlain by a red-brown deposit consisting of clayey silt till / residual soil, the top of which was encountered in the boreholes between about Elevation 133 m and 130 m (about 5 m to 8 m below the existing rail cut grade). The till / residual soil deposit was proved for a thickness of at least 2 m to 4 m in the boreholes. In Borehole H4, where the deposit was fully penetrated, the till / residual soil deposit is about 2.6 m thick.

The till / residual soil contains trace to some sand, and trace gravel and shale fragments; relatively thin layers or lenses of weathered shale and limestone were noted within this deposit in the samples recovered at this site and the adjacent Plains Road site. A grain size distribution test result obtained for a sample of this till / residual soil is shown on Figure 3.

Atterberg limits testing carried out on four samples of this till / residual soil measured plastic limits of 13 to 15 per cent, liquid limits of 22 to 31 per cent (but typically 22 to 27 per cent), and plasticity indices of 9 to 16 per cent. The results of the limits testing, shown on Figure 4, indicate that the till / residual soil is inorganic and of low plasticity. The measured natural moisture contents range from 8 to 12 per cent, typically at or slightly below the plastic limit for the material.

The red-brown till / residual soil has a hard consistency, with measured SPT 'N' values well above 100 blows per 0.3 m of penetration.

4.2.4 Shale Bedrock

Red-brown shale bedrock of the Queenston Formation was encountered below the red-brown till / residual soil in one borehole (Borehole H4) at this structure site. At this location, the surface of the shale was encountered at Elevation 129.7 m, approximately 8 m to 9 m below the rail cut grade and 16 m to 17 m below the existing Highway 6 grade. Shale bedrock was also encountered in one borehole at the Plains Road structure site immediately east of the Highway 6 – CP Rail structure site; in that borehole, the surface of the shale bedrock is at Elevation 124.9 m.

4.3 Groundwater Conditions

All of the boreholes were dry during and on completion of the drilling operations for this site. A piezometer was installed in Borehole H3, in the northwest area of the structure, and in two boreholes immediately east of Highway 6 at the Plains Road – CP Rail structure site. Each of these piezometers is screened within the till / residual soil. The water levels measured in the piezometers on November 11 and November 22, 2002 varied from about Elevation 138 m to 136 m, as summarized in the following table:

Borehole No.	***Piezometer Tip and Filter Pack Interval***	***Water Level Elevation***	
		Nov. 11, 2002	***Nov. 22, 2002***
H3	Till / residual soil below Elevation 132.1 m	138.0 m	138.0 m
P2	Till / residual soil below Elevation 127.6 m	136.0 m	136.0 m
P8	Till / residual soil below Elevation 126.1 m	137.1 m	136.9 m

These levels may reflect a groundwater table generally sloping downward toward the north and east. 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. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fintan J. Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of the report.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng. Associate	Fintan J. Heffernan, P.Eng. Designated MTO Contact

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 6 OVERPASS AT CP RAIL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-04

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed Highway 6 Overpass at CP Rail. The recommendations are based on interpretation of the factual data obtained from a limited number of 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 for design of the proposed overpass and staging of this construction. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project.

It is understood that Highway 6 will be widened from its existing four-lane configuration to six lanes, including a "tall wall" median; the increase in width will be about 15 m. Widening or replacement of the existing three-span Highway 6 – CP Rail overpass structure is therefore necessary. According to the general layout drawing dated November 1963, the existing three-span structure is supported on 3.2 m to 3.5 m wide spread footings, with the north abutment founded at about Elevation 142 m, the piers founded at about Elevation 136.6 m, and the south abutment founded at about Elevation 140.5 m. It is understood that the preferred option involves replacement of the existing structure with a single-span structure.

6.2 Bridge and Retaining Wall Foundation Options

At the structure site, the natural ground surface varies from about Elevation 145 m to 140 m, generally declining toward the south and west. The CP Rail line has been constructed in a cut between 2 m and 6 m deep, with the rail grade at about Elevation 138.8 m to 138 m within the proposed structure limits; the rail grade and the cut depth decline toward the west. Highway 6 has been constructed on embankment fill, with its profile grade at about Elevation 146.5 m to 147 m within the limits of the structure. The existing CP Rail overpass approach embankments are up to about 6 m in height.

The native soils at the site consist of hard clayey silt to silty clay till, overlying hard clayey silt till / residual soil below about Elevation 133 m to 130 m (about 5 m to 8 m below the existing rail cut grade), in turn overlying shale bedrock. The native till soils at relatively shallow depth below the existing rail cut are suitable for support of the proposed abutments and associated retaining walls, such as concrete cantilever retaining walls, on shallow foundations. In addition, the native soils are suitable for use of a mechanically-reinforced soil retaining wall system (retained soil system or RSS wall) adjacent to the bridge abutments at this site.

Deep foundations, such as driven steel H-piles, could also be considered for support of the proposed single-span structure and associated retaining walls.

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

6.3 Spread Footings

The bridge abutments may be supported on spread footings placed below any topsoil and fill to be founded within the undisturbed clayey silt to silty clay till deposit. Any associated concrete cantilever wing walls / retaining walls may also be supported on spread footings founded on the undisturbed clayey silt to silty clay till deposit.

Based on the current General Arrangement drawing, the new north and south abutments for the Highway 6 replacement structure will cross over a portion of the north and south piers of the existing structure, respectively. The founding level for the existing piers is understood to be about Elevation 136.6 m, based on the November 1963 General Layout drawing for the existing three-span structure; however, the current General Arrangement drawing for the replacement structure shows the existing piers to be founded deeper, at about Elevation 135.8 m. It is assumed that the existing footings will be removed. It is recommended, therefore, that within the footprint of the existing pier foundations, the new footings extend to at least the existing pier founding level to ensure that they are founded on undisturbed native soil.

Outside of the existing pier footprints, the new footings for the Highway 6 replacement structure could be stepped upward to minimize excavation requirements. A founding level of Elevation 137 m may be taken for the design of the spread footings outside of the footprint of the existing piers. As noted in Section 6.3.3, a minimum of 1.2 m of soil cover must be provided above the footing level to ensure adequate protection against frost penetration. In this regard, consideration could be given to stepping the footing upward toward the east end of the structure, where the rail cut grade is higher.

Consideration must be given to the construction staging sequence to ensure that, where the existing structure remains in place during construction, the existing footings are not undermined by the new foundations.

6.3.1 Geotechnical Resistance

Spread footings placed on the undisturbed clayey silt to silty clay till deposit, at or below the design elevations given above, may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 700 kPa. The settlement of the footings will be dependent on the footing size, configuration, and applied loads. The geotechnical resistance at Serviceability Limit

States (SLS) may be taken as 450 kPa. These geotechnical resistances assume a footing width of 4.2 m and a footing length of about 45 m. The geotechnical resistances should be reviewed if there are significant changes in the foundation geometry.

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

6.3.2 Resistance to Lateral Loads

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

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 Driven Steel H-Pile Foundations

Steel H-piles should be driven to found at least 1 m into the “100-blow” clayey silt till / residual soil deposit. The surface of this deposit was encountered in the boreholes between about Elevation 133 m and 130 m, but typically at about Elevation 132 m. It should be recognized that the hard nature of the till will likely result in heavy driving; in addition, the use of driven foundations at this site must take into account the potential presence of cobbles and boulders within the deposits. The piles should be equipped with driving shoes for protection. If the H-piles meet refusal above approximately Elevation 132 m such that the pile length is inadequate for structural considerations, the pile would have to be withdrawn and augering carried out to remove or displace the obstruction, prior to re-driving. Pre-augering could be employed to ensure a reasonable pile length without undue heavy driving, depending on the type of pile-driving equipment used for construction.

6.4.1 Axial Geotechnical Resistance

The factored axial resistance at ULS for steel HP 310 x 110 piles driven to found within the clayey silt till / residual soil, as described above, may be taken as 1,600 kN. The settlement of the individual piles and the pile group at the above pile loads 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.

To achieve the above design resistance of 1,600 kN at ULS, the piles should be driven to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of approximately 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap selected piles to confirm the set after adjacent piles have been driven, in accordance with MTO's current Special Provision.

6.4.2 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equation for cohesive soils:

$$k_h = \frac{k_{s1}}{5B}$$

where B is the pile diameter (m) and

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

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

Soil Unit	$\boldsymbol{k_{s1}}$
Very Stiff to Hard Clayey silt above about Elevation 132 m	50 to 100 MPa/m
Very Stiff Silty Clay below about Elevation 132 m	100 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:

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

6.4.3 Frost Protection

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

6.5 Retained Soil System (RSS) Walls

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

Use of an RSS wall is considered appropriate for the proposed wing walls / retaining walls, which will be between about 4 m and 7 m high. For the reinforced earth mass founded on the hard clayey silt to silty clay till below the existing rail cut grade (i.e. at or below about Elevation 138.5 m to 138 m at the east and west limits of the proposed structure, respectively), the factored geotechnical resistance at ULS will depend on the width of the reinforced soil mass and the following values may be used for design:

- 275 kPa for a 4 m high wall; and
- 450 kPa for a 7 m high wall.

These values assume that the RSS wall acts as a unit and utilizes the full width of the reinforced soil mass which is taken as two-thirds of the height of the wall. The geotechnical resistance at SLS, for 25 mm of settlement, may be taken as 400 kPa.

The resistance to lateral forces / sliding resistance between the compacted Granular "A" and the till subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, between the compacted Granular "A" of the RSS wall and the generally hard clayey silt to silty clay till 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.

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.

6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the walls:

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC)*.

- For Case I, the pressures are based on the 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.

6.7 Excavations and Temporary Roadway Protection

Excavations for construction of the abutment and wing wall / retaining wall footings will extend to at least 1.2 m below the existing rail cut grade. This will require excavation into the existing embankment side slopes for Highway 6, which is about 9 m above the rail cut grade, and into the existing permanent cut slopes for the rail, which are up to about 6 m high. The excavations will extend through the existing embankment and abutment foreslope fill where present, and into hard clayey silt to silty clay till. The excavations will generally extend to about 1 m below the groundwater level at the site; within the footprint of the existing pier footings, excavations may extend deeper below the groundwater level. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and the till are classified as Type 3 and 1 soils, respectively, according to the OHSA.

Temporary excavations (i.e. those which are only open for a relatively short period) through the existing embankment fill materials should be made with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V). Temporary excavation side slopes within the clayey silt to silty clay till should be maintained no steeper than 1H:1V.

Where space restrictions preclude the use of temporary open cut excavations, temporary roadway protection will be required. The temporary excavation support system should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in SP105S19.

6.8 Approach Embankment Design

The widening of the Highway 6 approach embankments and replacement of the existing three-span structure with a shorter single-span structure will require placement of up to 6 m of fill adjacent to the existing embankment sideslopes and atop the existing abutment foreslopes. It is assumed that the abutment walls and footings for the existing three-span structure will be partially or fully removed prior to construction of the approach embankments, to avoid creating a "hard point" beneath the embankments that could affect the performance of the pavements.

Based on the borehole results, the embankment subgrade soils will consist of existing fill and very stiff to hard clayey silt till. Any topsoil, organic matter and softened / loosened soils should be stripped from below the widening and existing abutment foreslope areas, and all subgrade soils should be proof-rolled prior to fill placement. The embankment fill should be placed and compacted in accordance with MTO's Special Provision SP105S10. 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.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, embankments up to about 6 m in height with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated slope instability. To reduce surface water erosion, placement of topsoil and seeding or pegged sod is recommended.

Settlement of the existing embankment comprised of clayey silt fill is expected to be negligible. For the new embankment areas, provided that the subgrade is properly prepared, any settlement is likely to occur within the new embankment fill itself. The amount of settlement will vary according to the thickness of new fill placed to construct the new embankment. This settlement will be differential with respect to the existing embankment. In order to minimize differential settlement between the new embankment areas and the existing embankment, the use of granular fill is recommended for the new construction. The majority of the settlement of granular fills will occur during construction, whereas the majority of settlement of cohesive fills, if used, would occur post-construction.

The new embankment fill should be keyed by benching into the existing embankment side slopes as well as into the existing abutment foreslopes, where these are left in place, to reduce the impact of differential settlement. Benching should be carried out in accordance with OPSD 208.01.

6.9 Design and Construction Considerations

6.9.1 Obstructions

The native soils at the site are glacially-derived and, as such, are expected to contain cobbles and boulders. In addition, the existing embankment is comprised of clayey silt fill that is likely reworked till; obstructions such as cobbles or boulders should, therefore, be anticipated within this fill. 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). Ultimately, provision will have to be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

6.9.2 Groundwater and Surface Water Control

Groundwater seepage into the excavation is expected to occur from within the fill (perched atop the clayey silt to silty clay till deposit), and from lenses or interlayers of permeable material that may be present within the 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.

The soils in which the footing or pile cap 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.

7.0 CLOSURE

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

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.	Fintan J. Heffernan, P.Eng.
Associate	Designated MTO Contact

LCC/FJH/lcc
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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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)
A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

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LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
l_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 = σ'_p/σ'_{vo}

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

* density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

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RECORD OF BOREHOLE No H1

METRIC

PROJECT 001-1141F

W.P. 19-95-00 LOCATION N 4,795,469.1 E 272,214.6 ORIGINATED BY GM

DIST Central HWY 6 BOREHOLE TYPE Continuous Split-Spoon Sampling COMPILED BY LCC

DATUM Geodetic DATE Oct.15/02 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
145.2	GROUND SURFACE					
0.0	Topsoil	1	SS	4	145	
144.9 0.3	Clayey Silt, trace sand, gravel, shale fragments, cinders and rootlets (FILL) Stiff to very stiff Brown Moist	2	SS	8, 12, 9		
		3	SS		144	o
		4	SS	15, 19, 21		o
		5	SS	23, 33	143	
142.6	Clayey Silt, trace sand, gravel and shale fragments (TILL) Hard Brown to grey-brown Dry to moist	6	SS	25/.15		o
142.3 2.9	END OF BOREHOLE					

Notes:
1. Borehole dry on completion of drilling operations.

2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on these logs to reflect the values that would be obtained using a standard-weight hammer.

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

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

RECORD OF BOREHOLE No H2

METRIC

PROJECT 001-1141F

W.P. 19-95-00 LOCATION N 4,795,420.9 E 272,258.9 ORIGINATED BY GM

DIST Central HWY 6 BOREHOLE TYPE Continuous Split-Spoon Sampling COMPILED BY LCC

DATUM Geodetic DATE Oct.16/02 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
144.7	GROUND SURFACE					
144.4 / 0.0	Topsoil					
0.3	Clayey Silt, trace sand, gravel and shale fragments (FILL) Stiff to very stiff Brown Moist	1	SS	3, 7, 9	144	
		2	SS	15, 16		o
		3	SS	20, 22, 21	143	
		4	SS	23, 23	142	o
		5	SS	22, 20, 15	141	
		6	SS	15, 15		o
		7	SS	15, 20, 33	140	
139.2 / 5.6	Clayey Silt, trace sand, gravel and shale fragments (TILL) Hard Brown to grey-brown Dry to moist	8	SS	23/.15		o
	END OF BOREHOLE					

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

Notes:
1. Borehole dry on completion of drilling operations.

2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on these logs to reflect the values that would be obtained using a standard-weight hammer.

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

Golder Associates

RECORD OF BOREHOLE No H3

METRIC

PROJECT 001-1141F

W.P. 19-95-00 — LOCATION N 4,795,456.6 E 272,230.5 — ORIGINATED BY GM

DIST Central HWY 6 — BOREHOLE TYPE 108mm Diameter Solid Stem Augers — COMPILED BY LCC

DATUM Geodetic — DATE Jan.23/01 — CHECKED BY ASP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
141.2	GROUND SURFACE					
0.0	Clayey Silt, trace sand and gravel (Fill) Very stiff to hard Brown				141	
		1	SS	19	140	
	Contains pieces of black slag between 1.5m and 2.0m depth.	2	SS	35		
139.2 / 2.0	Clayey Silt to Silty Clay, trace to some sand, trace gravel and shale fragments (Till) Hard Brown to grey-brown Moist				139	
		3	SS	37		
		4	SS	40/.05	138	
		5	SS	95	137	
		6	SS	85		0 4 59 37
					136	
		7	SS	80	135	
					134	
		8	SS	100/.23		
					133	
132.7 / 8.5	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil) Hard Red-brown Dry to moist					
		9	SS	95/.15	132	
					131	
130.4 / 10.8	END OF BOREHOLE	10	SS	100/.15		

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

Notes:

1. Borehole dry on completion of drilling operations.
2. Water level in piezometer measured on November 11, 2002 at 3.2m depth (Elev.138.0m) .
3. Water level in piezometer measured on November 22, 2002 at 3.3m depth (Elev.137.9m) .

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

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RECORD OF BOREHOLE No H4

METRIC

PROJECT 001-1141F

W.P. 19-95-00

DIST Central HWY 6

DATUM Geodetic

LOCATION N 4,795,464.5 E 272,256.9

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE Jan.23/01

ORIGINATED BY GM

COMPILED BY LCC

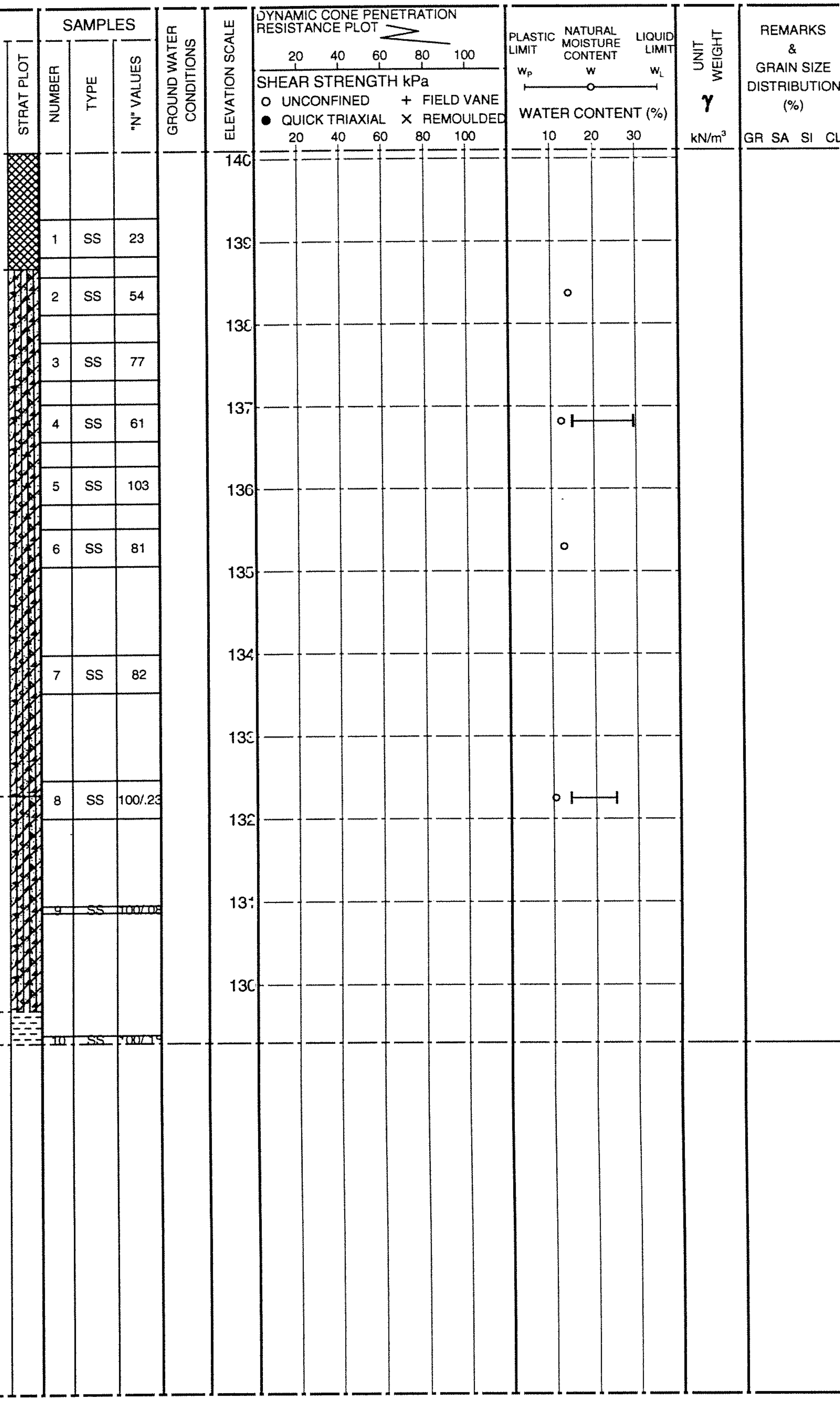
CHECKED BY ASP

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
140.1	GROUND SURFACE
0.0	Clayey Silt, trace sand and gravel (Fill) Very stiff Brown
138.7 / 1.4	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till) Hard Grey-brown Dry to moist
132.3 / 7.8	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil) Hard Red-brown Dry to moist
129.7	Shale (Bedrock) Red-brown
129.3 / 10.8	END OF BOREHOLE Note: Borehole dry on completion of drilling operations.

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	23
2	SS	54
3	SS	77
4	SS	61
5	SS	103
6	SS	81
7	SS	82
8	SS	100/.23
9	SS	100/.08
10	SS	100/.1[illegible]



$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No P1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

DIST Central HWY 6

DATUM Geodetic

LOCATION N 4,795,502.9 E 272,225.0

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE Jan.08/01

ORIGINATED BY GM

COMPILED BY LCC

CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
145.1	GROUND SURFACE			
0.0	Clayey Silt, trace sand and gravel Very stiff Red-brown	1	SS	21
143.7 / 1.4	Clayey Silt, trace to some sand, trace gravel (Till) Hard Brown becoming grey-brown below 3m depth Dry to moist	2	SS	69
		3	SS	86
		4	SS	100
		5	SS	90
		6	SS	80
		7	SS	75/.15
		8	SS	50
		9	SS	66
		10	SS	75
		11	SS	70
132.0 / 13.1	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil) Hard Red-brown Dry to moist	12	SS	72/.15

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (145 to 131); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No P1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4,795,502.9 E 272,225.0

ORIGINATED BY GM

DIST Central HWY 6

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

COMPILED BY LCC

DATUM Geodetic

DATE Jan.08/01

CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	-- CONTINUED FROM PREVIOUS PAGE --										
	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil) Hard Red-brown Dry to moist		13	SS	102/.15		130				
							129				
128.1			14	SS	110/.23						
17.0	END OF BOREHOLE Note: Borehole dry on completion of drilling operations.										

$+^3, \times^3$: Numbers refer to Sensitivity

$O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No P4

METRIC

PROJECT 001-1141F

W.P. 19-95-00

DIST Central HWY 6

DATUM Geodetic

LOCATION N 4,795,470.0 E 272,252.4

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE Jan.24/01

ORIGINATED BY GM

COMPILED BY LCC

CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
140.1	GROUND SURFACE				
0.0	Clayey Silt, some sand, trace gravel and rootlets, Stiff, Brown	1	SS	11	
138.7 / 1.4	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till), Hard, Brown to grey-brown, Dry to moist	2	SS	110	
		3	SS	70	
		4	SS	90	
		5	SS	90	
		6	SS	80	
		7	SS	91	
		8	SS	100/.28	
132.2 / 7.9	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil), Hard, Red-brown, Dry to moist	9	SS	105/.15	4 18 60 18
		10	SS	100/.1?	
129.3 / 10.8	END OF BOREHOLE				

Note:
Borehole dry on completion of drilling operations.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; ELEVATION SCALE 140 139 138 137 136 135 134 133 132 131 130; STRAT PLOT.

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

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RECORD OF BOREHOLE No P5

METRIC

PROJECT 001-1141F

W.P. 19-95-00

DIST Central HWY 6

DATUM Geodetic

LOCATION N 4,795,491.6 E 272,248.5

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

DATE Jan.22/01

ORIGINATED BY GM

COMPILED BY LCC

CHECKED BY ASP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
138.7	GROUND SURFACE						
0.0	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till) Hard Brown to grey-brown Dry to moist				138		
		1	SS	69			
		2	SS	87	137		
		3	SS	72	136		
		4	SS	86		~12 (range to ~28)	
		5	SS	75	135		
		6	SS	90	134	~11	
					133		
		7	SS	88	132		
		8	SS	100/.15	131	~15 (range ~13 to ~31)	5 13 56 26
130.2 / 8.5	Clayey Silt, trace to some sand, trace gravel and shale fragments (Till/Residual Soil) Hard Red-brown Dry to moist				130		
		9	SS	100/.05	129		
127.9 / 10.8		10	SS	102/.15	128	~9	

END OF BOREHOLE

Note:
Borehole dry on completion of drilling operations.

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

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

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

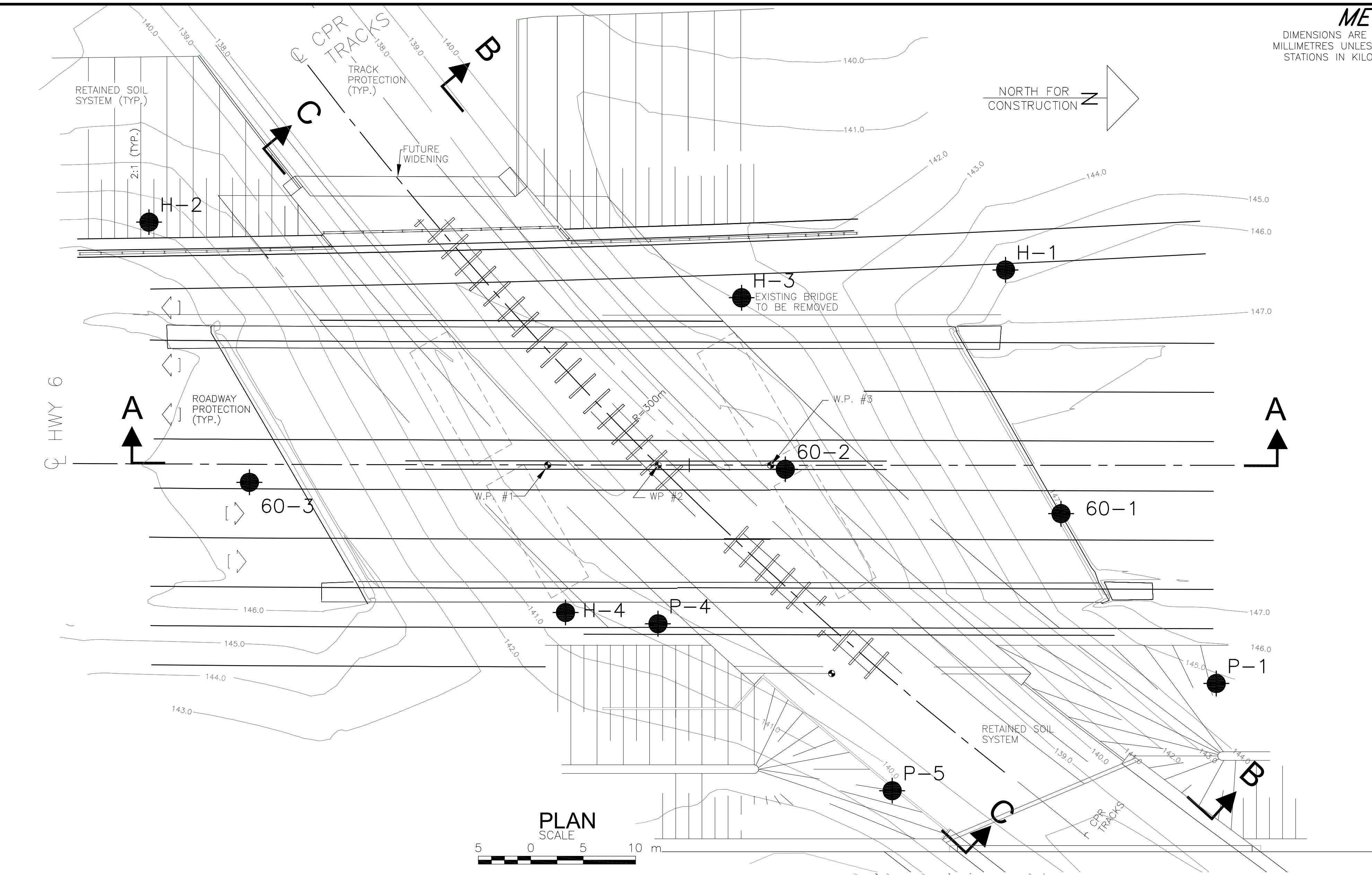
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 19-95-04

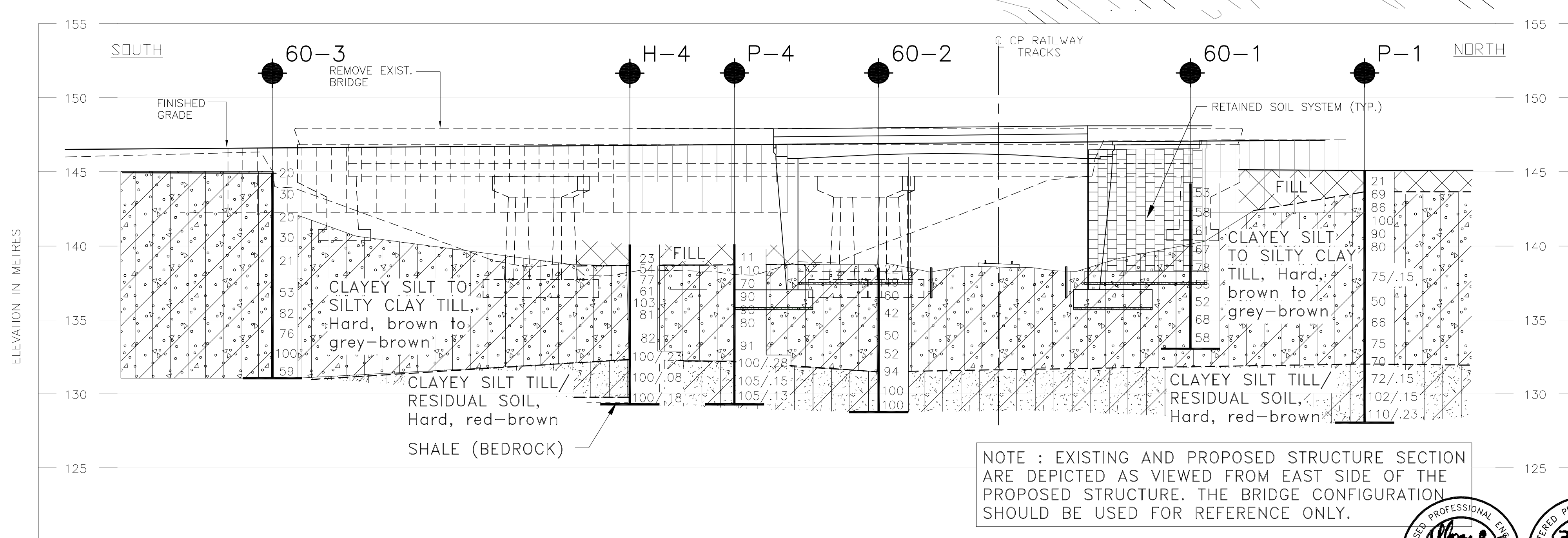
HIGHWAY 6 OVERPASS
AT CP RAIL
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

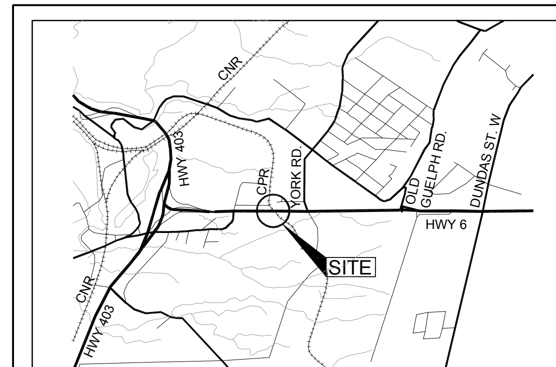


PLAN
SCALE
5 0 5 10 m



NOTE : EXISTING AND PROPOSED STRUCTURE SECTION ARE DEPICTED AS VIEWED FROM EAST SIDE OF THE PROPOSED STRUCTURE. THE BRIDGE CONFIGURATION SHOULD BE USED FOR REFERENCE ONLY.

SECTION A : PROFILE ALONG CENTRELINE OF HIGHWAY 6
SCALE
5 0 5 10 m



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)
- 100 — Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
H-1	145.2	4795469.1	272214.6
H-2	144.7	4795420.9	272258.9
H-3	141.2	4795456.6	272230.5
H-4	140.1	4795464.5	272256.9
P-1	145.1	4795502.9	272225.0
P-4	140.1	4795470.0	272252.4
P-5	138.7	4795491.6	272248.5
60-1	144.2	4795485.4	272224.5
60-2	138.5	4795468.3	272237.2
60-3	144.9	4795440.5	272267.3

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.

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 was provided in digital format by URS Canada Inc., File name "2002-04/underpass.dwg"

LICENSED PROFESSIONAL ENGINEER L.C. COYNE PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F.J. HEFFERNAN PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY.	PROJECT NO. 001-1141F		DIST.
SUBM'D. LCC	CHKD. LCC	DATE: APRIL 2005	SITE:
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 1

PLOT DATE: August 28, 2006
FILENAME: T:\Projects\2000\001-1141F\JULY2006\P1141FCP001.dwg

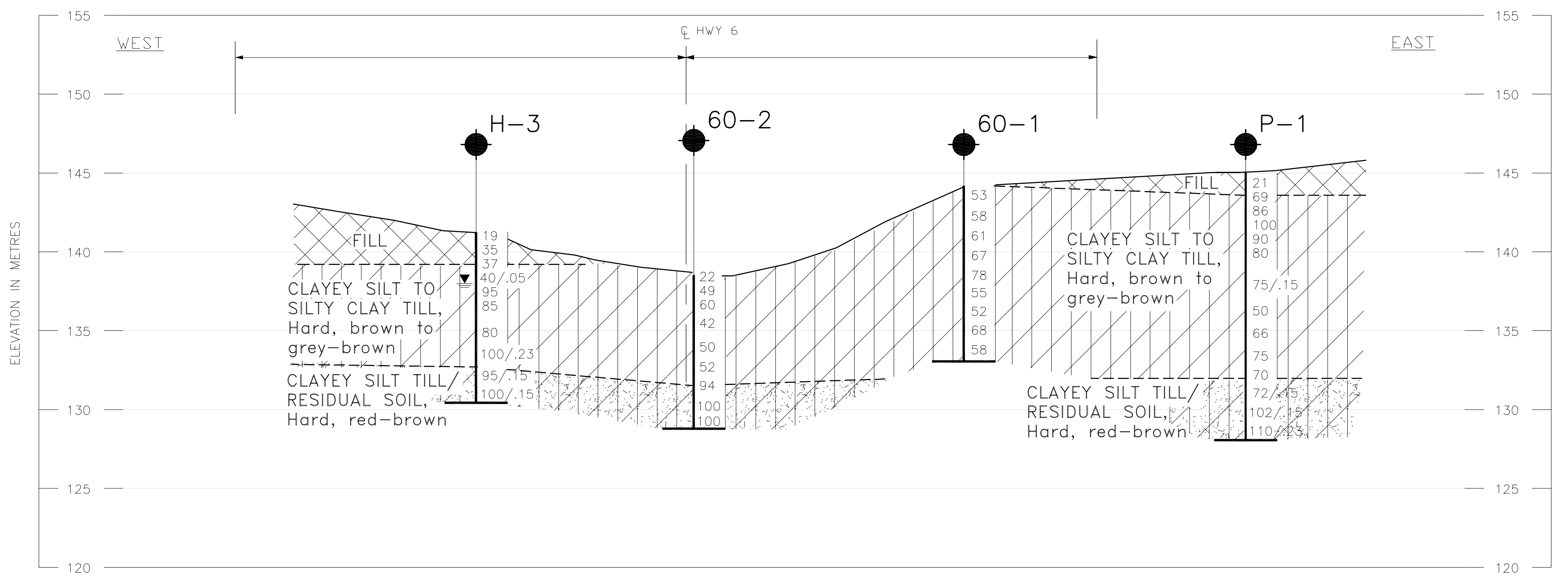
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 19-95-04

HIGHWAY 6 OVERPASS
AT CP RAIL
SOIL STRATA

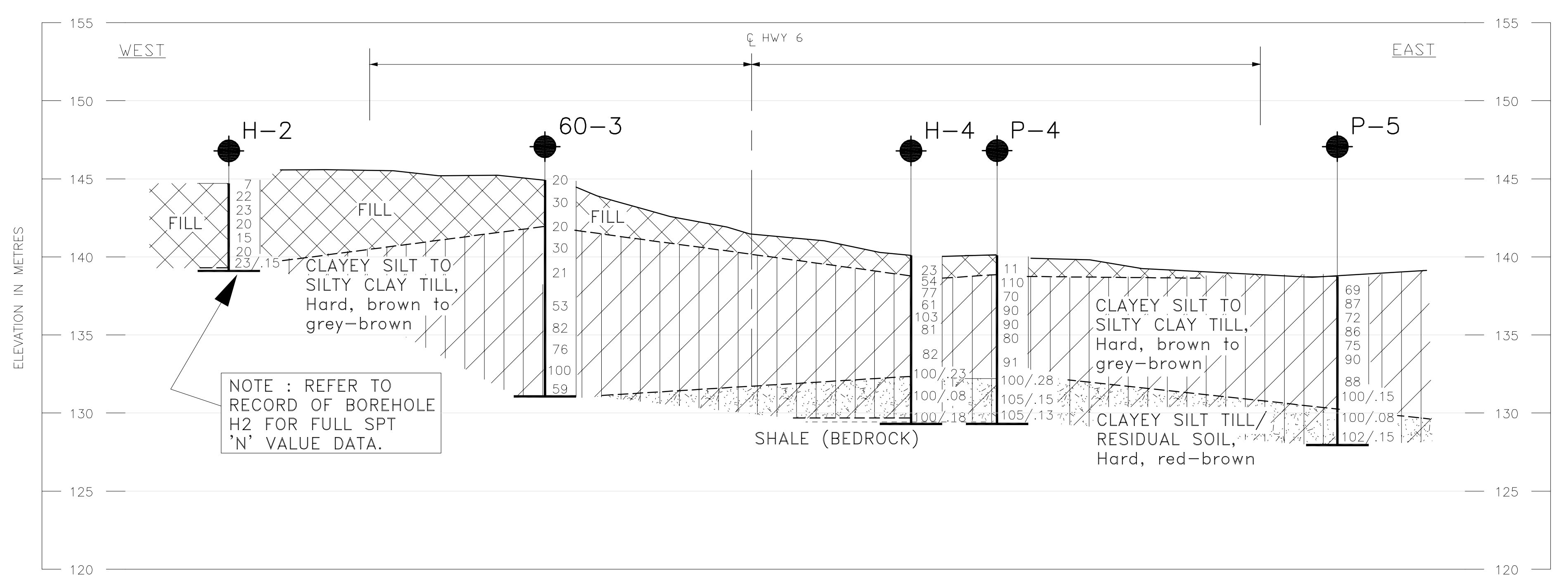
SHEET

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



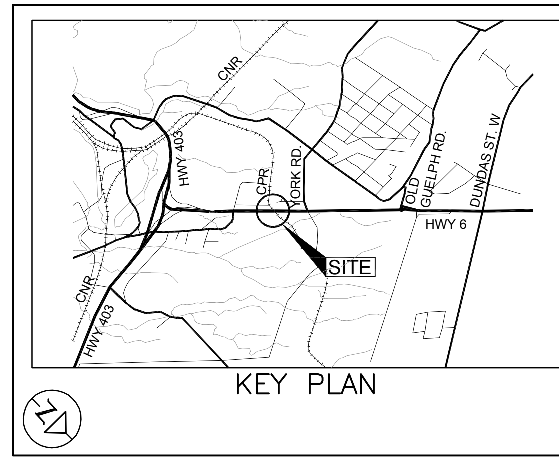
SECTION B : PROFILE ALONG NORTH ABUTMENT

SCALE
5 0 5 10 m



SECTION C : PROFILE ALONG SOUTH ABUTMENT

SCALE
5 0 5 10 m



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)
- 100 — Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
H-1	145.2	4795469.1	272214.6
H-2	144.7	4795420.9	272258.9
H-3	141.2	4795456.6	272230.5
H-4	140.1	4795464.5	272256.9
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60-1	144.2	4795485.4	272224.5
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NOTES

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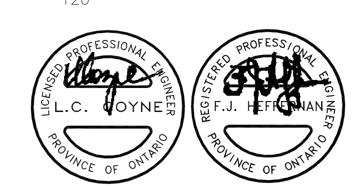


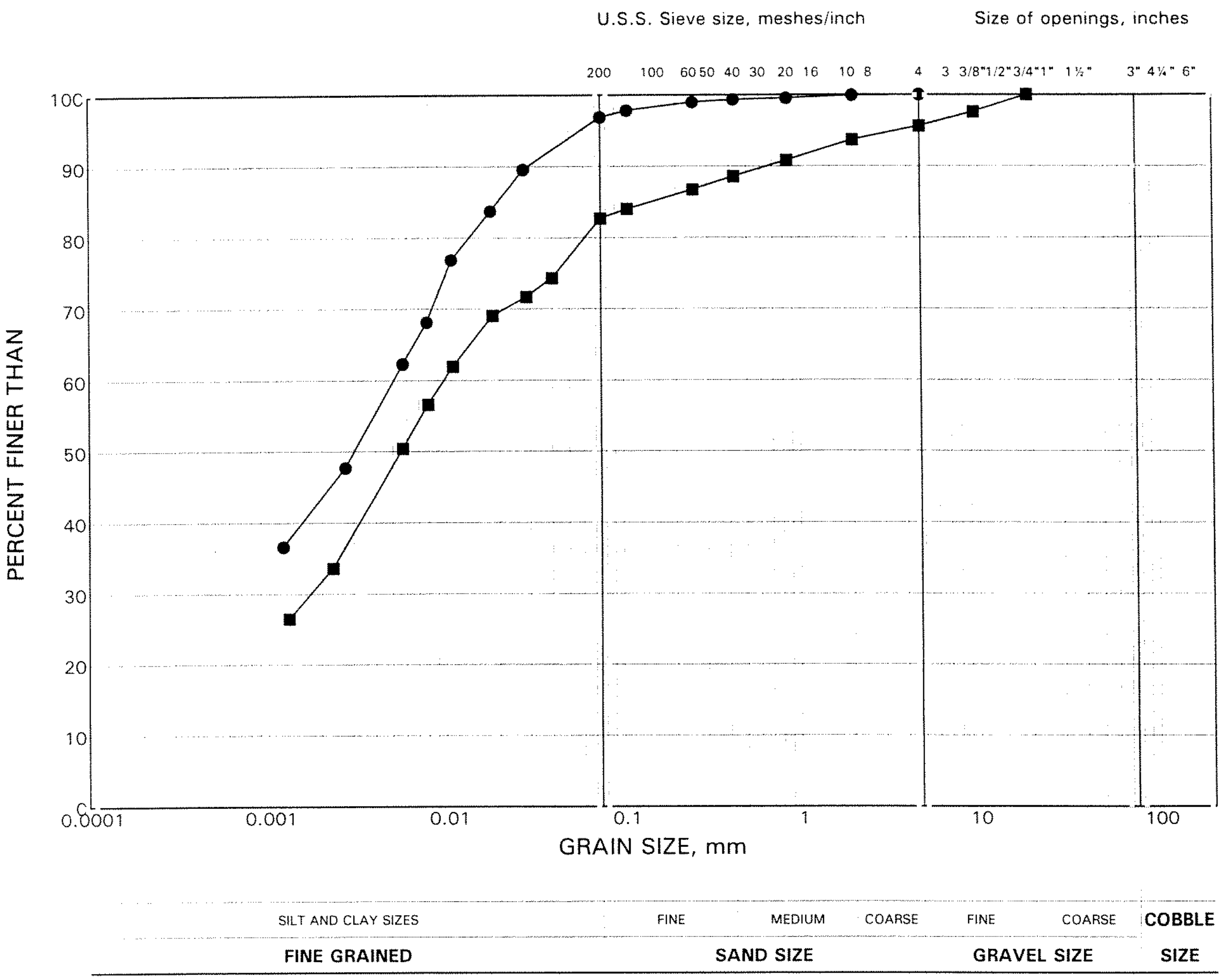
NO.	DATE	BY	REVISION

Geocres No.

HWY.	PROJECT NO. 001-1141F		DIST.
SUBM'D. LCC	CHKD. LCC	DATE: APRIL 2005	SITE:
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 2

GRAIN SIZE DISTRIBUTION TEST RESULTS
Clayey Silt to Silty Clay Till

FIGURE 1



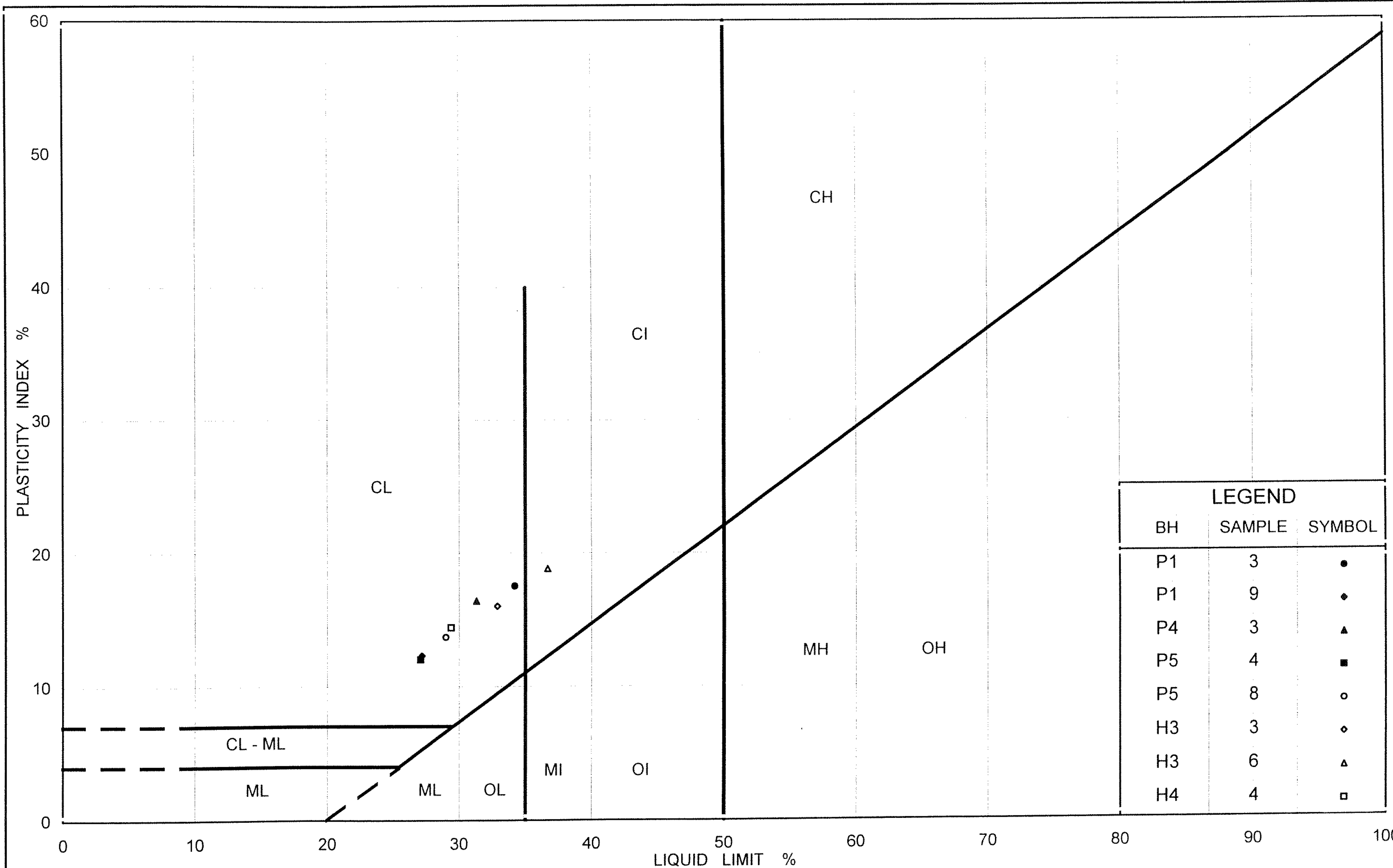
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	H3	6	136.4
■	P5	8	130.8

Project 001-1141F

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation

Ontario

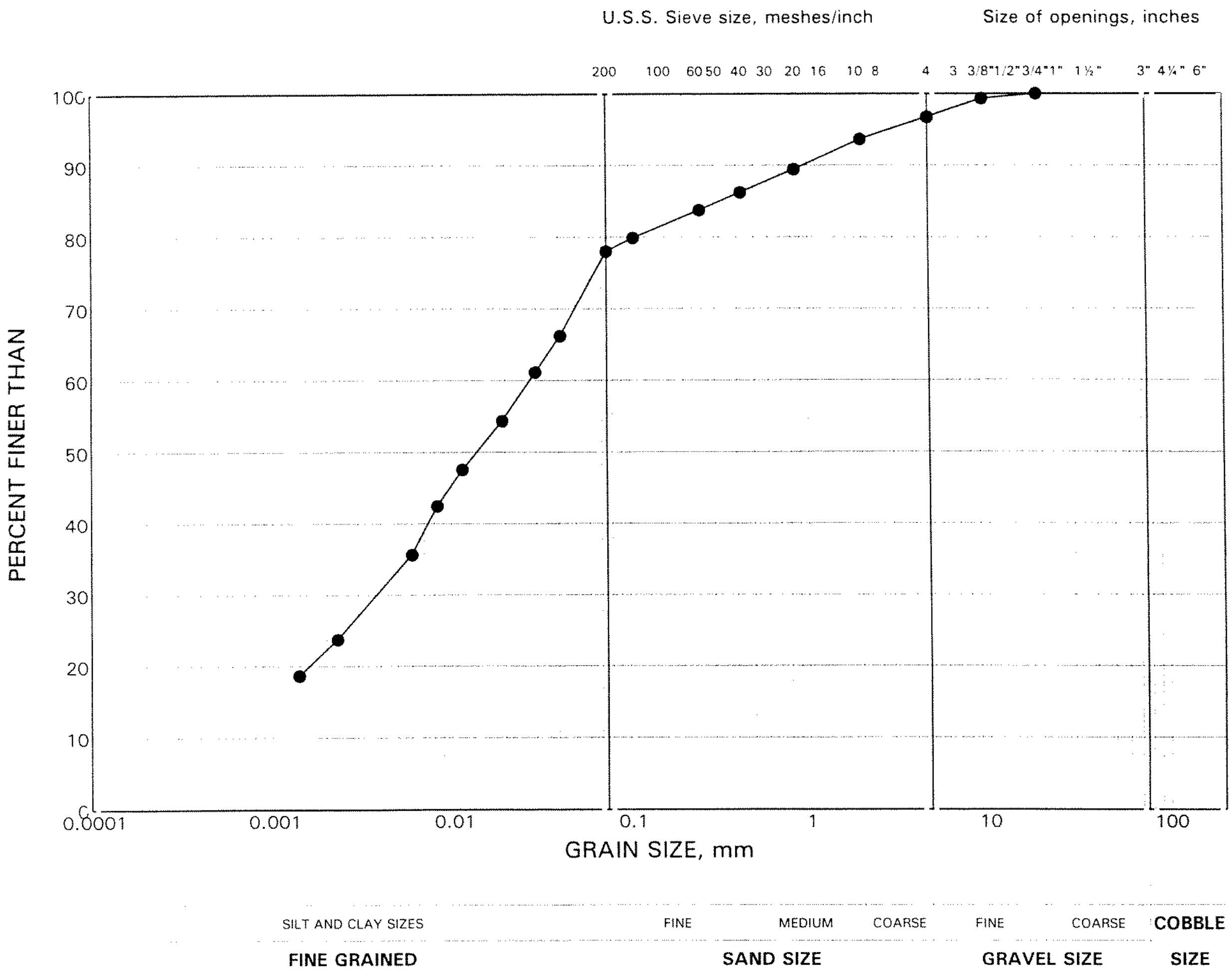
PLASTICITY CHART
Clayey Silt to Silty Clay Till

FIG No. 2

Project No. 001-1141F

GRAIN SIZE DISTRIBUTION TEST RESULTS
Clayey Silt Till / Residual Soil

FIGURE 3

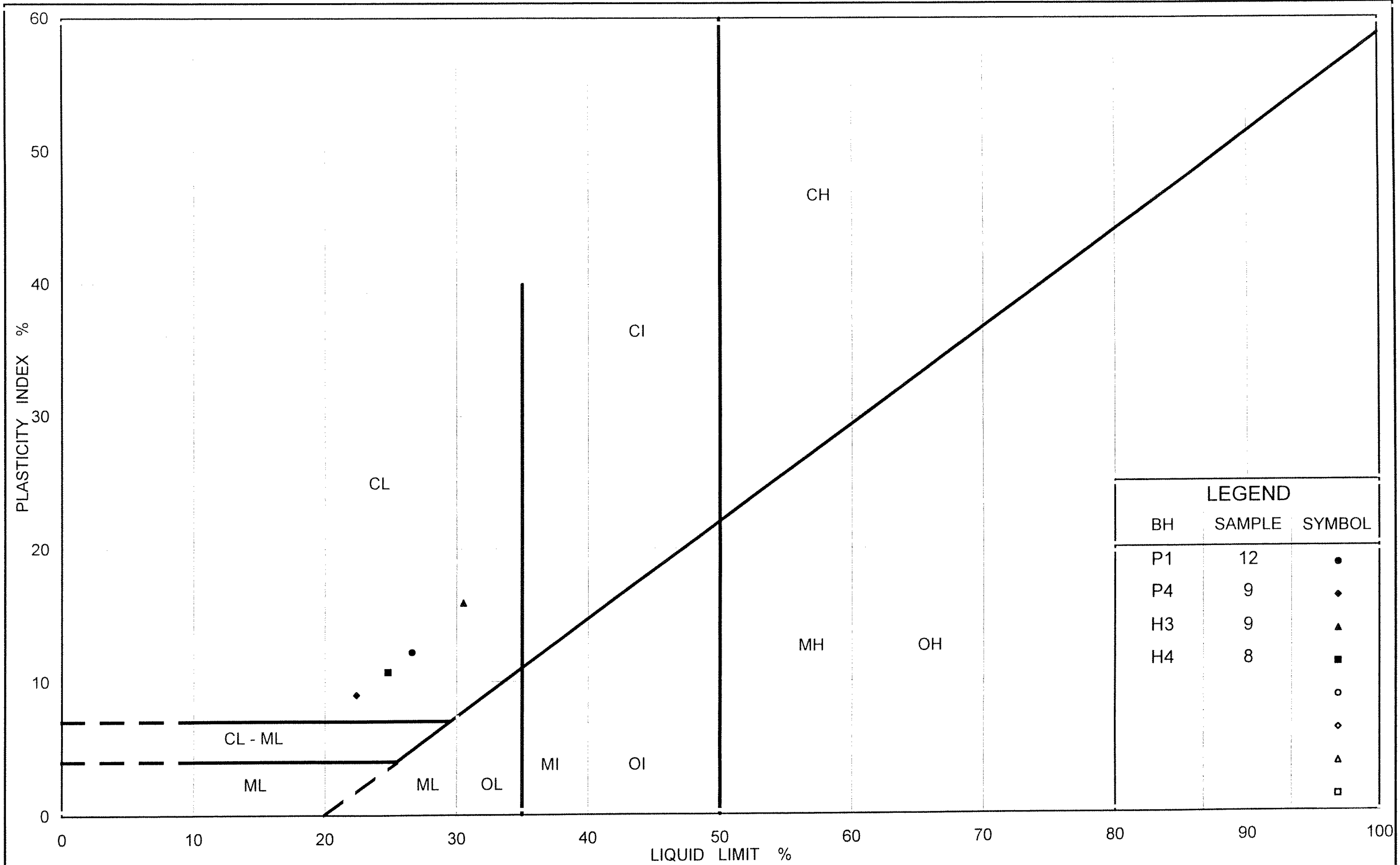


LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	P4	9	130.8

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation Ontario	PLASTICITY CHART Clayey Silt Till / Residual Soil	FIG No. 4 Project No. 001-1141F

APPENDIX A

RECORDS OF BOREHOLES FROM 1960 SUBSURFACE INVESTIGATION
(BOREHOLES 60-1, 60-2 AND 60-3)

Order No. 0-10-5

Enclosure No. 2

BOREHOLE 60-1

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 1.

Date: 13-17 OCT. 1960.

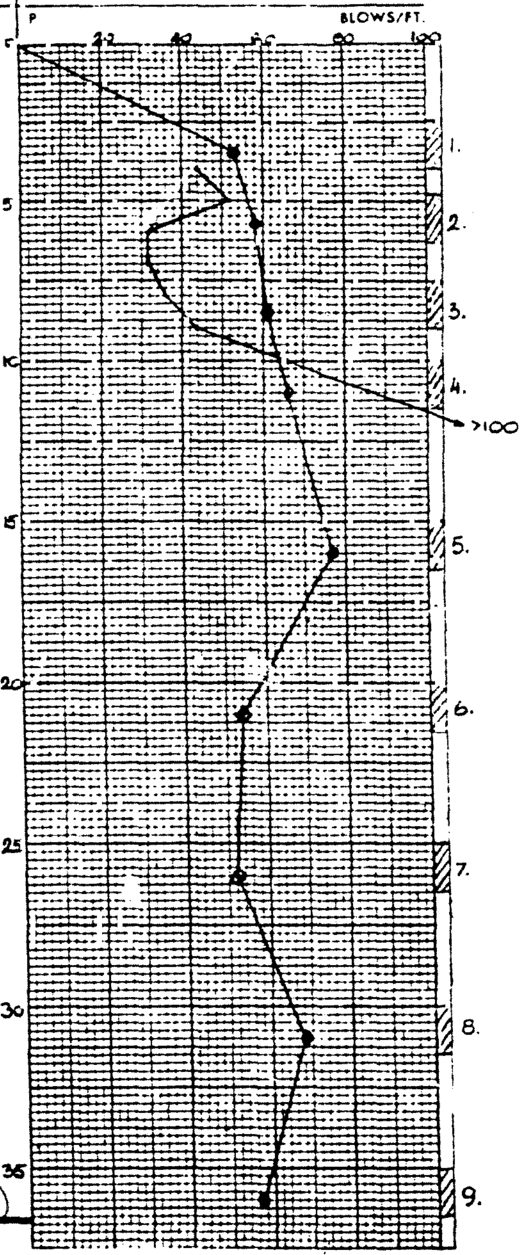
Project: HWY #6 - C.P.R. OVERHEAD
Location: 1/2 MI. S. OF CLAPPISONS CORNERS
Hole Location: SEE ENCLOSURE NO. 1.
Hole Elevation and Datum: 473.1 FT.
Field Supervisor: J.P. Prep.: J.P.
Driller: R.R. Checked: L.R.S.

LEGEND
Shear Strength (C)
Unconfined compression ⊕
Vane test and sensitivity (S) +S
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	GROUND SURFACE.	473.1	
	ORGANIC TOPSOIL.	(El. 144.2m)	
	BROWN CLAY OF INTERMEDIATE PLASTICITY CONTAINING APPR. 5% SAND AND FINE GRAVEL MAX 3/4"Ø (CLAY TILL)	470 460 450 440 (El. 133.1m)	5 10 15 20 25 30 35
	END OF BOREHOLE	436.6'	

STRENGTH AND PENETRATION RESISTANCE



Order No. O-10-5

Enclosure No. 3.

BOREHOLE 60-2

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: #2

Date: 18-19 OCT. 1960.

Project: HWY #6 - C.P.R. OVERHEAD
Location: 1.2 MI. S. OF CLAPPISONS CRNS.
Hole Location: SEE ENCLOSURE NO. 1.
Hole Elevation and Datum: 454.5 FT
Field Supervisor: J.P. Prep.: J.P.
Driller: R.R. Checked: L.R.S.

LEGEND

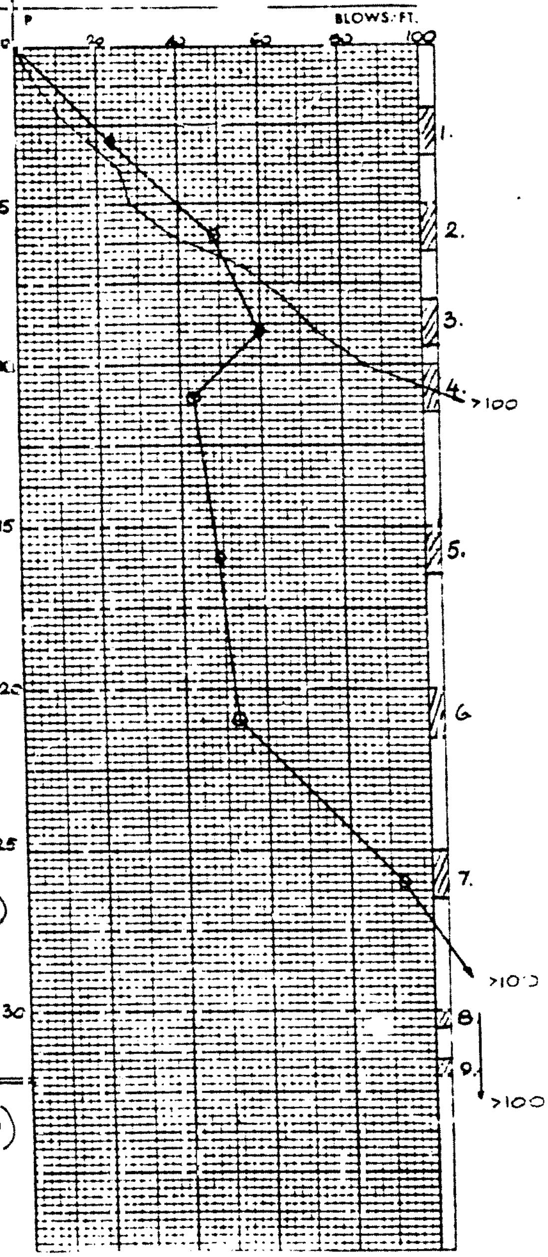
Shear Strength (C)
- Unconfined compression ⊕
- Vane test and sensitivity (S) +S

Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	GROUND SURFACE	454.5 (138.5m)	0
	GRASS AND ROOTS		
	BROWN CLAY OF INTERMEDIATE PLASTICITY CONTAINING APPR. 5% SAND AND FINE GRAVEL MAX. 3/4" ϕ (CLAY TILL)	450	5
			10
		440	15
			20
	TURNS REDDISH	(El. 132.1m)	
		430	25
	MIXED WITH BROKEN RED & GREEN SHALE.	427.1 (El. 130.5m)	
		420	30
	END OF BOREHOLE	422.5 (El. 128.8m)	

STRENGTH AND PENETRATION RESISTANCE



Order No. Q-10-5 Enclosure No. 4.

BOREHOLE 60-3

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: # 3.

Date: 20-21 OCT. 1960

Project: HWY #6 - C.P.R. OVERHEAD
Location: 1.2 MI. S. OF CLAPPISONS CORNERS
Hole Location: SEE ENCLOSURE NO. 1.
Hole Elevation and Datum: 475.5 FT.
Field Supervisor: J.P. Prep.: J.P.
Driller: R.R. Checked: L.R.S.

LEGEND

Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)

Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	GROUND SURFACE	475.5 (El. 144.9m)	0
	FILL (BROWN CLAY TILL)	470	5
		467.0 (El. 142.3m)	
	BROWN CLAY OF INTERMEDIATE PLASTICITY CONTAINING APPR. 5% SAND AND FINE GRAVEL MAX. 3/4" ϕ (CLAY TILL)	460	15
			20
		450	25
			30
		440	35
	END OF BOREHOLE	439.0 (El. 133.8m)	

STRENGTH AND PENETRATION RESISTANCE