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**FOUNDATION
INVESTIGATION AND DESIGN REPORT
BRUCE TRAIL PEDESTRIAN TUNNEL
HIGHWAY 6 WIDENING
BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-07**

Submitted to:

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PART A

**FOUNDATION INVESTIGATION REPORT
BRUCE TRAIL PEDESTRIAN TUNNEL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-07**

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 Bruce Trail pedestrian tunnel under Highway 6, immediately south of Old Guelph Road. 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 blasted on either side of Highway 6. Below the crest, the ground surface declines from about Elevation 215 m to 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 that is up to about 15 m in height.

The proposed Bruce Trail pedestrian tunnel site is located at Station 12+320, immediately south of Old Guelph Road. At this location, the existing Highway 6 grade is at about Elevation 189 m to 188 m, and the highway embankment is between 8 m and 10 m high relative to the surrounding natural grade. The existing embankment side slopes are oriented at about 1.5 horizontal to 1 vertical (1.5H:1V).

3.0 INVESTIGATION PROCEDURES

A total of four boreholes have been drilled as part of the subsurface investigation program for the proposed Bruce Trail pedestrian tunnel. One of the boreholes (Borehole E1) was drilled on the west shoulder of the existing highway in November 2000, just to the south of the proposed tunnel location, as part of the high embankment investigation and preliminary tunnel alignment study. This borehole was advanced using a truck-mounted BOA-5M drill rig supplied and operated by Groundworks Drilling Company Ltd. of Weston, Ontario.

In October and November 2002, after selection of the pedestrian tunnel location and approval by the Niagara Escarpment Commission and Bruce Trail personnel, three additional boreholes (Boreholes B1 to B3) were drilled along the proposed alignment at about Station 12+320. Borehole B2 was advanced on the east side of the existing highway using a truck-mounted CME-75 drill rig supplied and operated by Geo-Environmental Drilling Ltd. of Milton, Ontario. Boreholes B1 and B3 were drilled at the west and east limits of the proposed pedestrian tunnel, respectively, on the existing escarpment / embankment side slopes using portable drilling equipment supplied and operated by Sonic Soil Sampling Ltd. of Concord, Ontario..

Boreholes B2 and E1 were drilled using solid stem augers to depths of about 12.5 m and 13 m, respectively, to extend through the existing embankment fill and into the underlying clayey silt till. In these boreholes, 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. In Boreholes B1 and B3, 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 two boreholes were advanced to about 3 m and 4 m depth respectively, to extend through existing fill, topsoil and loose / soft surficial deposits, into the underlying clayey silt till. The water levels in the open boreholes were observed throughout the drilling operations.

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

The borehole locations were determined by Golder Associates relative to points staked by Callon Dietz on the proposed tunnel alignment and other features in the field. The ground surface elevations at the borehole locations were determined from the DTM model developed by Callon Dietz and URS for this project. 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 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 disturbed bedrock at the contact between the Halton Till and the shale.

4.2 Site Stratigraphy

The borehole locations and ground surface elevations for the four boreholes advanced at this site 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 6. 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 boreholes drilled through the existing road embankment in the vicinity of the proposed pedestrian culvert encountered up to about 9 m to 10 m of variable embankment fill, ranging in composition and consistency from compact silty sand to gravel, to firm to hard clayey silt to silty clay. The existing fill is generally underlain by hard clayey silt till. Outside of the existing highway embankment footprint, at the west end of the proposed pedestrian tunnel, surficial topsoil, clayey silt and sand and silt deposits are present atop the clayey silt till. Toward the east end of the proposed pedestrian tunnel, the existing fill comprising the residential driveway embankment is underlain by loose to compact, silty sand alluvium associated with the Grindstone Creek valley; this alluvium in turn overlies hard clayey silt till.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections, and a stratigraphic profile along this section of the highway is shown on Drawing 1.

4.2.1 Embankment Fill

Boreholes E1 and B2, drilled through the existing Highway 6 embankment, encountered about 9 m to 10 m of variable embankment fill, with the base of the fill at about Elevation 178 m to 180 m at the borehole locations. Existing fill related to the residential driveway embankment on the east side of Highway 6 was also encountered in Borehole B3, near the eastern limit of the proposed pedestrian culvert.

The upper 4 m of the fill encountered in Borehole E1, located on the west edge of the highway, consists of gravel and cobbles with some sand and silt. Below this, and below the road base materials encountered in Borehole B2, the fill consists of firm to hard clayey silt containing trace to some sand, gravel, shale and limestone fragments, and compact sandy gravel to silty sand. Cobbles and/or boulders were inferred within the fill during drilling, based on grinding of the augers and/or resistance to penetration of the sampler. The results of grain size distribution testing carried out on samples of the embankment fill are shown on the attached Figure 1; it is noted that the size of the split-spoon sampler limits the size of the recovered particles to less than about 45 mm diameter.

Atterberg limits testing carried out on three samples of the clayey silt embankment fill measured plastic limits of 16 to 17 per cent, liquid limits of 27 to 32 per cent, and plasticity indices of 11 to 15 per cent. The results of this limit testing, shown plotted on Figure 2, indicate that the clayey silt fill is inorganic and of low plasticity. The measured natural water contents in samples of the clayey silt fill ranged from 6 to 17 per cent, but were typically between about 10 and 15 per cent.

In the surficial gravel / cobble fill encountered in Borehole E1, and in the zone of granular fill encountered at depth in Borehole B2, the measured Standard Penetration Test (SPT) 'N' values range from 14 to 27 blows per 0.3 m of penetration, indicating that the granular portion of the fill has a compact relative density. The measured SPT 'N' values in the clayey silt portion of the embankment fill ranged from 5 to 38 blows per 0.3 m of penetration, indicating a variable consistency ranging from firm to hard. Higher SPT 'N' values of about 42 to 66 blows per 0.3 m of penetration were measured, but it is considered that these results were affected by the presence of gravel and/or cobbles in the sample zone.

4.2.2 Surficial Clayey Silt and Sand to Sand and Silt

Outside of the existing highway embankment footprint, at the west end of the proposed pedestrian tunnel, Borehole B1 encountered about 0.7 m of topsoil overlying a 0.8 m thick layer of clayey

silt containing trace to some sand, trace gravel, and silty sand pockets. One Atterberg limit test carried out within this layer measured a plastic limit of 18 per cent, a liquid limit of 34 per cent, and a plasticity index of 16 per cent; these results, shown on Figure 3, indicate that the surficial clayey silt is inorganic and of low plasticity. The measured SPT 'N' values within this layer range from 2 to 8 blows per 0.3 m of penetration, indicative of a soft to stiff consistency.

Immediately below the surficial clayey silt in Borehole B1 at the west end of the proposed pedestrian tunnel, a 0.3 m thick layer of moist to wet sand and silt is present. This material has a loose relative density, based on one measured SPT 'N' value of 9 blows per 0.3 m of penetration. At the east end of the proposed pedestrian tunnel, Borehole B3 encountered a 2 m thick layer of sand to silty sand alluvium, associated with the Grindstone Creek valley, below the existing residential driveway embankment fill. This alluvial material is loose to compact, based on measured SPT 'N' values of 7 to 22 blows per 0.3 m of penetration. Grain size distribution test results for a sample of the sand to sand and silt materials from the east and west limits of the proposed tunnel are shown on Figure 4.

4.2.3 Clayey Silt Till

The embankment fill and, where present, surficial clayey silt or silty sand to sand and silt deposits are underlain by a deposit of brown to red-brown clayey silt till. The surface of this till deposit was encountered in the boreholes along the proposed pedestrian tunnel alignment between Elevations 178 m and 182.6 m, rising from west to east.

The clayey silt till contains trace to some sand, gravel, shale and limestone fragments. Grain size distribution test results obtained on four samples of the till are shown on Figure 5 following the text of this report. Atterberg limits testing carried out on four samples of the clayey silt till measured plastic limits of 13 to 15 per cent, liquid limits of 24 to 31 per cent, and plasticity indices of about 10 to 15 per cent. The results of the limits testing, depicted on Figure 6, indicate that the clayey silt till is inorganic and of low plasticity. The measured natural moisture contents range from 12 to 15 per cent, and are typically at or slightly below the plastic limit for the material.

The till has a very stiff to hard consistency, with measured SPT "N" values ranging from 19 to 73 blows per 0.3 m of penetration, but is generally hard with the majority of the 'N' values greater than 30 blows per 0.3 m of penetration.

4.3 Groundwater Conditions

The water levels in the open boreholes were observed during the drilling operations and are summarized in the following table:

Borehole Number	Borehole Location	Water Level in Open Borehole	
		Depth	Elevation
B1	West limit of proposed tunnel	1.5 m	178.3 m
E1	West side of existing highway	Dry at 12.8 m	Dry at 176.1 m
B2	East side of existing highway	12.3 m	177.1 m
B3	East limit of proposed tunnel	Dry at 4.3 m	Dry at 181.8 m

In general, the water encountered in the open boreholes at the site is associated with the surficial sand to sand and silt deposit, such as was encountered in Borehole B1, or is contained in zones of granular fill perched within cohesive embankment fill or atop the native clayey silt till deposit, such as was encountered in Borehole B2.

It is anticipated that Grindstone Creek, located immediately east of and parallel to the Highway 6 embankment at the proposed pedestrian tunnel site, is the main influence for the groundwater at the site. The Grindstone Creek channel invert falls steeply to the south in this area, from about Elevation 190 m to 180 m, and the natural ground surface declines both to the south and the west. Given the topographic conditions, it is anticipated that the groundwater level at the pedestrian tunnel site will be in the range of Elevation 182 m to 178 m, falling toward the south and west.

However, 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.

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PART B

**FOUNDATION DESIGN REPORT
BRUCE TRAIL PEDESTRIAN TUNNEL
HIGHWAY 6 WIDENING BETWEEN HIGHWAYS 403 AND 5
W.P. 19-95-07**

6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed Bruce Trail pedestrian tunnel. 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 alternatives and to design the proposed pedestrian culvert. 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 the Bruce Trail pedestrian tunnel will cross under Highway 6 at Station 12+320, immediately south of Old Guelph Road and the existing Bruce Trail. In this area, the existing Highway 6 grade is between Elevation 189 m and 188 m, declining southward down the escarpment. Highway 6 itself has been constructed on embankment fill that is up to about 10 m high relative to the surrounding natural grade.

It is further understood that a shallow cut-and-cover excavation is the preferred option for the construction of the proposed pedestrian tunnel. The pedestrian tunnel is planned to consist of a 3.5 m high, 2.9 m wide concrete culvert with its invert at Elevation 185 m to 184.75 m, about 3.9 m to 4.3 m below the final Highway 6 grade. Based on the subsurface conditions encountered in the boreholes, it is anticipated that compact sand and gravel fill or very stiff clayey silt fill will be present at the proposed invert elevation where the box is within the limits of the existing embankment. The west end of the proposed culvert will be supported on up to 5 m of new fill, which is required to widen the highway embankment. The east end of the proposed culvert will be supported on existing embankment fill and/or compact alluvium; the topographic contours indicate that near the eastern limit of the tunnel, up to 1 m of new fill will be required to infill the existing Grindstone Creek valley after it is channelized and redirected.

6.2 Box Culvert Foundations

The existing embankment fill which will form the founding soils for the proposed pedestrian culvert, although variable, is considered suitable for the support of a concrete box culvert. The use of an open culvert supported on spread footings founded within the embankment fill is not recommended, due to the differential settlements that could arise as a result of the fill variability.

6.2.1 Geotechnical Resistances

It is understood that the proposed pedestrian culvert will be founded at about Elevation 185 m. For the variable embankment fill (including the existing fill and the proposed fill placed for

embankment widening – assumed to be granular fill as discussed in Section 6.5) that will comprise the founding soils at this site, a factored geotechnical resistance at Ultimate Limit States (ULS) of 250 kPa may be used for design of the box culvert. The geotechnical resistance at Serviceability Limit States (SLS), for 25 mm of settlement, may be taken as 200 kPa.

6.2.2 Bedding and Backfill

It is noted that there may be soft / loose zones within the existing fill and alluvium which will require subexcavation and replacement if exposed within the subgrade for the new pedestrian culvert. Due to the highly variable subgrade conditions anticipated along the length of the culvert, it is recommended that at least 500 mm of OPSS Granular “A” bedding should be provided under the culvert. The bedding should be compacted to at least 95 per cent of the Standard Proctor maximum dry density. The bedding material should be placed as soon as practicable after reaching the base of the excavation and following completion of inspection.

Backfill to the culvert should be in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 and 803.02. Select free-draining granular fill meeting the specifications of OPSS Granular “A” or Granular “B” but with less than 5 per cent passing the 200 sieve should be used as backfill behind the culvert walls. This fill should be placed and compacted in accordance with MTO Special Provision SP105S10. The fill depth during placement should be maintained equal on both sides of the culvert with one side not exceeding the other by more than 400 mm. It is noted that, where the culvert excavation intercepts rock fill within the existing embankment, a geotextile separator is recommended to prevent migration of the granular into the rock fill.

6.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the culvert 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:

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Other surcharge loadings should be accounted for in the design, as required.
- Backfill to the culvert (as described above) should be placed within the wedge-shaped zone defined by a line drawn upward and outward from the base of the culvert at 1 horizontal to 1 vertical. For the box culvert, it is assumed that the structural support does not allow lateral movement of the walls and therefore at-rest pressures should be used in the design. The lateral earth pressures acting on the walls will depend on the granular fill as placed; 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: At rest, K_o	0.43	0.43

- If the wall support allows lateral yielding (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure.

It should be noted that the above design recommendations and parameters assume level backfill and ground surface behind the culvert. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

6.4 Excavations and Temporary Roadway Protection

6.4.1 Temporary Open-Cut Excavations

Excavations for construction of the proposed Bruce Trail pedestrian tunnel will extend to about 4.5 m to 5 m below the existing highway grade to allow for placement of a granular pad prior to construction of the pedestrian culvert. The excavations are expected to extend through the road base materials into the existing variable embankment fill. Some “perched” groundwater should be anticipated within the road base materials, the surficial cohesionless fill where it is present, and any zones or layers of granular fill within the clayey silt fill. Groundwater and surface water control are addressed in Section 6.6.

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 embankment fill is classified as a Type 3 soils according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through the existing fill should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

6.4.2 Temporary Excavation Support

Where space restrictions and construction staging requirements preclude the use of temporary open-cuts, a temporary excavation support system will be required. A soldier pile and lagging system is considered to be the most appropriate form of temporary support for this site. 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 at this location should meet Performance Level 2 as specified in SP105S19.

6.5 Embankment Design and Construction

The widening of the Highway 6 embankment will require placement of up to about 10 m of fill. Based on the borehole results, the embankment subgrade soils outside the existing embankment will generally consist of surficial clayey silt to sand / silty sand, or very stiff to hard clayey silt till. The widening will also require fill placement within the existing Grindstone Creek valley on the east side of the highway where surficial soils are expected to be variable and will likely contain recent alluvium and organic deposits.

In order to minimize differential settlement of the pedestrian culvert where founded within the existing embankment and within the widened areas, it is recommended that granular material be used for embankment construction under both ends of the culvert. All topsoil, organic material and softened / loosened soils on the existing embankment side slopes, on the escarpment / valley slopes and on the valley floor within the widening footprint should be removed and all subgrade soils proof-rolled prior to placement of the new fill material. In addition, the new embankment fill must be keyed into the existing embankment side slopes in accordance with OPSD 208-01 in order to reduce the impact of differential settlement. 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.

Provided that the embankment subgrade is properly prepared, settlement of the existing and new embankment fill and the underlying soils is estimated to be between 25 mm and 50 mm. The majority of this settlement will take place during construction.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, 10 m high embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated slope instability. Mid-height benches, 2 m in width, should be provided where the embankment is equal to or greater than 8 m in height. To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.6 Design and Construction Considerations

6.6.1 Obstructions

The existing embankment is comprised predominantly of earth fill; however, there is still a proportion of rock (cobbles and boulders) within the fill. In addition, the underlying native till deposit is expected to contain cobbles and boulders, although none were specifically encountered in the boreholes drilled at the site. The presence of these obstructions will affect the installation of driven or augered steel H-piles for temporary excavation support, and will also affect the

installation of soil 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.

6.6.2 Groundwater and Surface Water Control

The Grindstone Creek channel is present within the footprint of the proposed pedestrian tunnel near its east limit. It is understood that the existing creek will be channelized within a storm sewer in the immediate vicinity of the pedestrian tunnel, and that the creek channel will be shifted eastward to accommodate the widened Highway 6 embankment. It is assumed that the new storm sewer construction and rechannelization of Grindstone Creek will be carried out prior to the construction of the new Bruce Trail pedestrian tunnel; otherwise, creek and surface water control will be required during construction.

Excavations for the Bruce Trail pedestrian tunnel will be maintained above the groundwater level; however, seepage into the excavation could occur from perched water within the existing granular road base fill, zones of granular embankment fill, or surficial sand to sand and silt layers. It is considered that the quantity of water seepage can be handled by pumping from properly filtered sumps placed at the base of the excavation. The sumps should be maintained outside the foundation limits.

The clayey silt fill soils in which the pedestrian culvert excavation 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 prior to placement of the bedding material, if the bedding material cannot 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.

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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 B1				1 OF 1		METRIC					
W.P. 19-95-00		LOCATION N 4,796,226.2 E 271,421.8				ORIGINATED BY GM									
DIST Central HWY 6		BOREHOLE TYPE Continuous Split-Spoon Sampling				COMPILED BY LCC									
DATUM Geodetic		DATE Oct. 15/02				CHECKED BY ASP									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N* VALUES			SHEAR STRENGTH kPa							
179.8 0.0	GROUND SURFACE Topsoil		1	SS	4 2 2	∇	179	<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div>			10 20 30	10 20 30	10 20 30	GR SA SI CL	
179.1 0.7	Clayey Silt, trace to some sand, trace gravel, containing silty sand pockets		2	SS	6 8										
178.3 1.5	Soft to stiff Mottled brown to red-brown Moist to wet		3	SS	9 19										
178.0 1.8	Sand and Silt, trace gravel Loose Moist to wet		4	SS	35 42										
177.0 2.8	Clayey Silt, some sand, trace to some gravel, shale and limestone fragments (Till) Very stiff to hard Brown to red-brown Moist END OF BOREHOLE				15/02		177								
<p>Notes:</p> <p>1. Water level in open borehole on completion of drilling at 1.5m depth (Elev. 178.3m).</p> <p>2. Borehole advanced using portable drilling equipment with a half-weight hammer. The SPT "N" values have been adjusted on this log to reflect the values that would be obtained using a standard-weight hammer.</p>															

ON_MOT 0011141F.GPJ ON_MOT.GDT 19/12/02

RECORD OF BOREHOLE No B2

1 OF 1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4,796,254.2 E 271,445.1

ORIGINATED BY PKS

DIST Central HWY 6

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

COMPILED BY LCC

DATUM Geodetic

DATE Oct. 18/02

CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV	DEPTH	DESCRIPTION	NUMBER	TYPE	N° VALUES			20	40	60	80	100			
189.4	0.0	GROUND SURFACE													
		Asphalt													
		Sand and Gravel (Fill)													
188.8	0.6	Moist													
		Clayey Silt, some sand and gravel, trace organics (Fill)	1	SS	15										
188.2	1.2	Stiff to very stiff													
		Brown/black													
		Moist													
		Clayey Silt to Silty Clay, some sand, some gravel, shale, limestone and siltstone fragments (Fill)	2	SS	19										
		Firm to very stiff													
		Red-brown to reddish-grey	3	SS	8										
		Moist to wet													
			4	SS	6										
			5	SS	7										
		Spoon bouncing on gravel/cobble in sample 6	6	SS	42										
183.9	5.5	Gravel, some sand and silt, trace clay to Silty Sand, some gravel, trace clay, containing clayey silt pockets (Fill)	7	SS	14										
		Compact													
		Brown to red-brown													
		Moist													
			8	SS	27										
180.1	9.3	Clayey Silt, some sand, trace to some gravel, shale and limestone fragments (Till)	9	SS	39										
		Hard													
		Red-brown to reddish-grey													
		Moist													
			10	SS	33										
176.8	12.6	END OF BOREHOLE	11	SS	38										
		Note:													
		1. Water level in open borehole on completion of drilling at 12.3m depth (Elev. 177.1m)													

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

CN_MOT_0011141F.GPJ ON_MOT.GDT 19/12/02

RECORD OF BOREHOLE No B3

1 OF 1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4,796,266.8 E 271459.9

ORIGINATED BY GM

DIST Central HWY 6


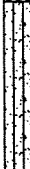

BOREHOLE TYPE Continuous Split-Spoon Sampling

COMPILED BY LCC

DATUM Geodetic

DATE Oct.11/02

CHECKED BY ASP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	w _p w w _L	WATER CONTENT (%)	10 20 30		
186.1 0.0	GROUND SURFACE Clayey Silt, some sand, trace to some gravel (Fill) Firm to very stiff Mottled red-brown to green Dry		1	SS	5 13 19									
			2	SS	16 8									
184.6 1.5	Silty Sand to Sand, some silt, trace gravel Loose to compact Brown to red-brown Dry becoming moist below 2.3m depth		3	SS	7 12 18									2 74 13 11
			4	SS	22 19									
182.6 3.5	Clayey Silt with sand to some sand, some gravel (Till) Very stiff to hard Brown to red-brown Moist		5	SS	20 33 28									
181.8 4.3	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 this log to reflect the values that would be obtained using a standard-weight hammer.		6	SS	39									4 20 46 30

RECORD OF BOREHOLE No E1

1 OF 1

METRIC

PROJECT 001-1141F

W.P. 19-95-00

LOCATION N 4,796,235.1 E 271,437.6

ORIGINATED BY AS/GM

DIST Central HWY 6

BOREHOLE TYPE 108mm Diameter Solid Stem Augers

COMPILED BY LCC

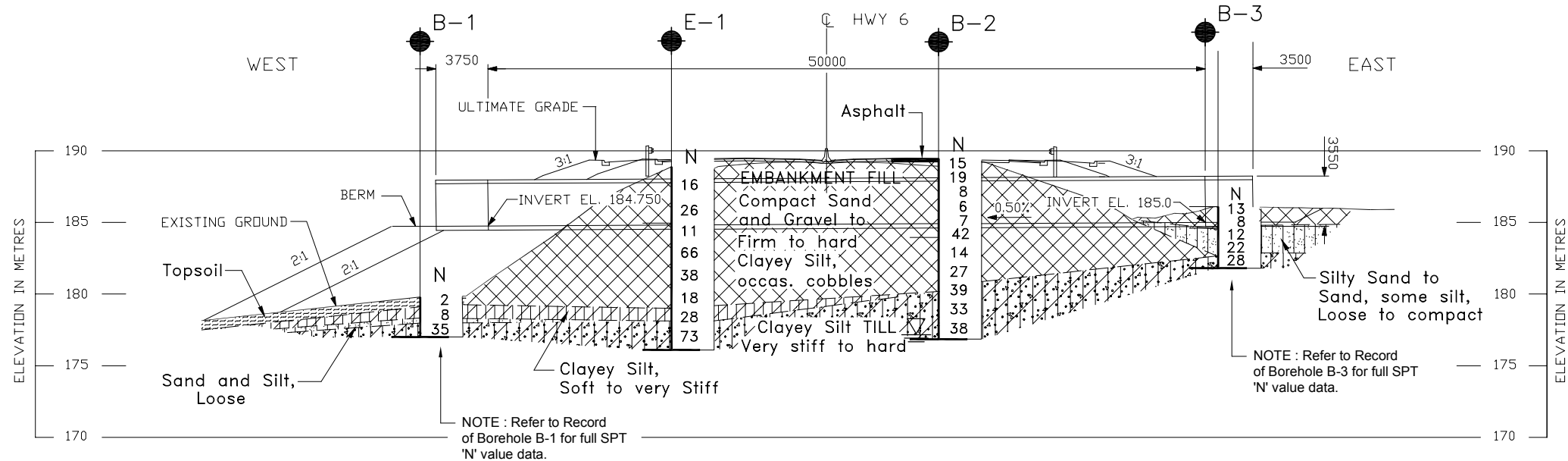
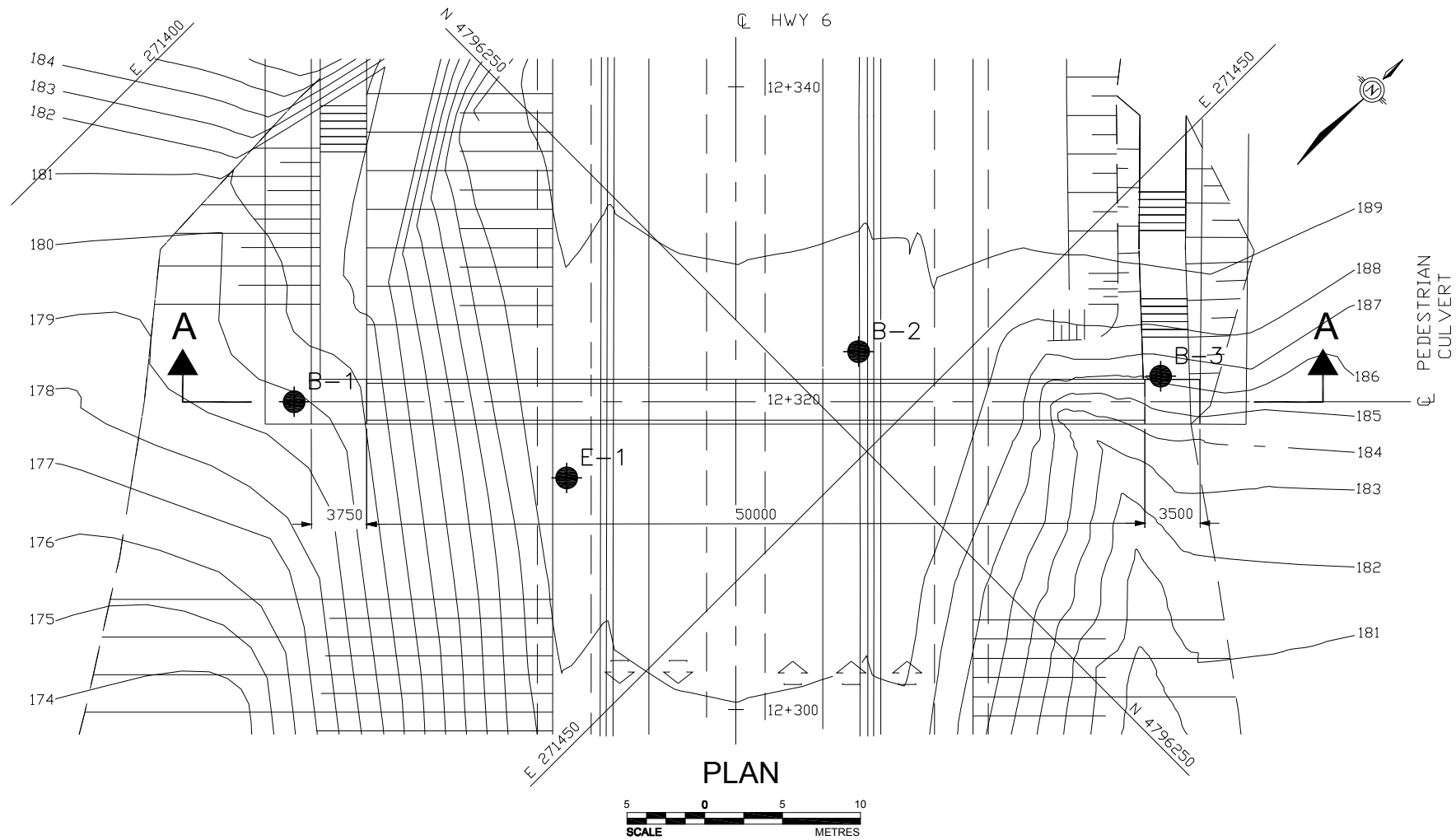
DATUM Geodetic

DATE Nov.21&23/00

CHECKED BY ASP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	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					W _p	W	W _L		
188.9 0.0	GROUND SURFACE Gravel and cobbles, some sand and silt (Fill) Compact Brown Dry													
			1	SS	16									
			2	SS	26									
184.9 4.0	Clayey Silt, trace to some sand, gravel and shale fragments (Fill) Stiff to hard Brown Moist Cobbles/boulders inferred from 5.2m to 6.1m and from 6.7m to 7.6m due to slow advance and grinding of augers.		3	SS	11									
			4	SS	66									
			5	SS	38									27 17 34 22
			6	SS	18									
178.7 10.2	Clayey Silt, trace sand, gravel and rootlets Very stiff Brown Moist		7	SS	28									3 19 48 30
178.1 10.8	Clayey Silt, some sand, trace gravel (Fill) Very stiff to hard Brown Moist		8	SS	73									
176.1 12.8	END OF BOREHOLE Note: Borehole dry on completion of drilling operations.													

ON MOT 0011141F.GPJ ON MOT.GDT 19/12/02



SECTION A-A, ALONG CENTRELINE OF PROPOSED PEDESTRIAN TUNNEL



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

CONT No.
WP No. 19-95-07

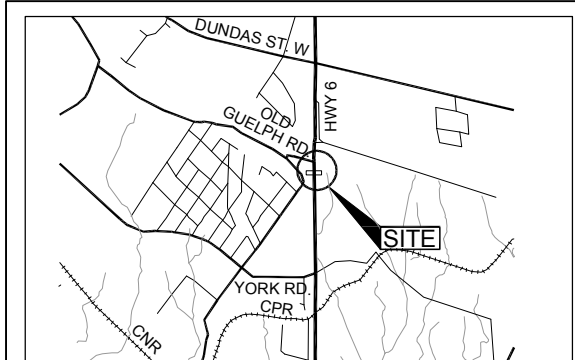
BRUCE TRAIL TUNNEL
STATION 12+320, HWY 6
BOREHOLE LOCATIONS & SOIL STRATA



SHEET



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

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
B-1	179.8	4796226.2	271421.8
B-2	189.4	4796254.2	271445.1
B-3	186.1	4796266.8	271459.9
E-1	188.9	4796235.1	271437.6

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"

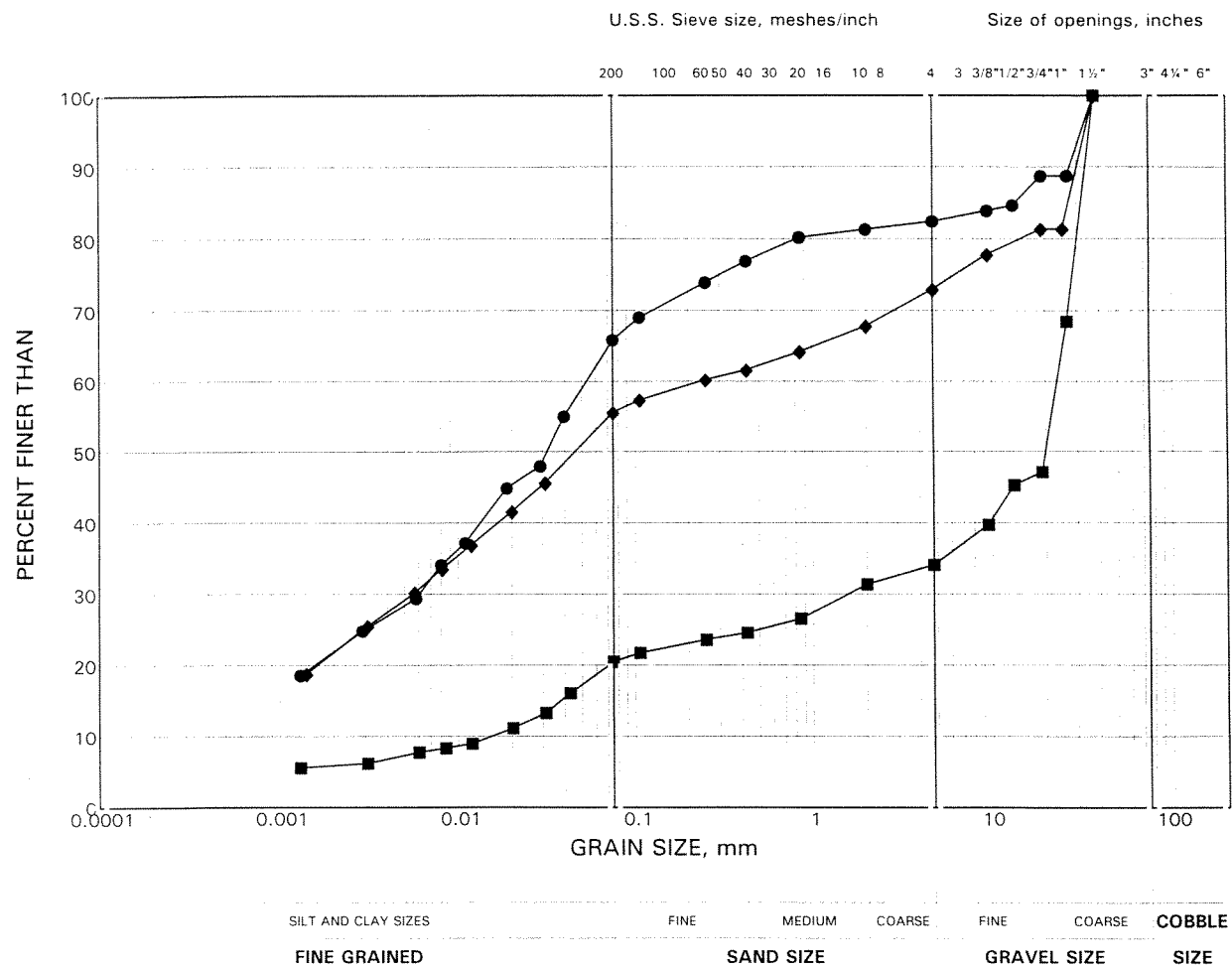


NO.	DATE	BY	REVISION
Geocres No.			
HWY.	PROJECT NO. 001-1141F		DIST.
SUBM'D. LCC	CHKD. LCC	DATE: APRIL 2005	SITE:
DRAWN: PS	CHKD. LCC	APPD. ASP	DWG. 1

GRAIN SIZE DISTRIBUTION TEST RESULTS

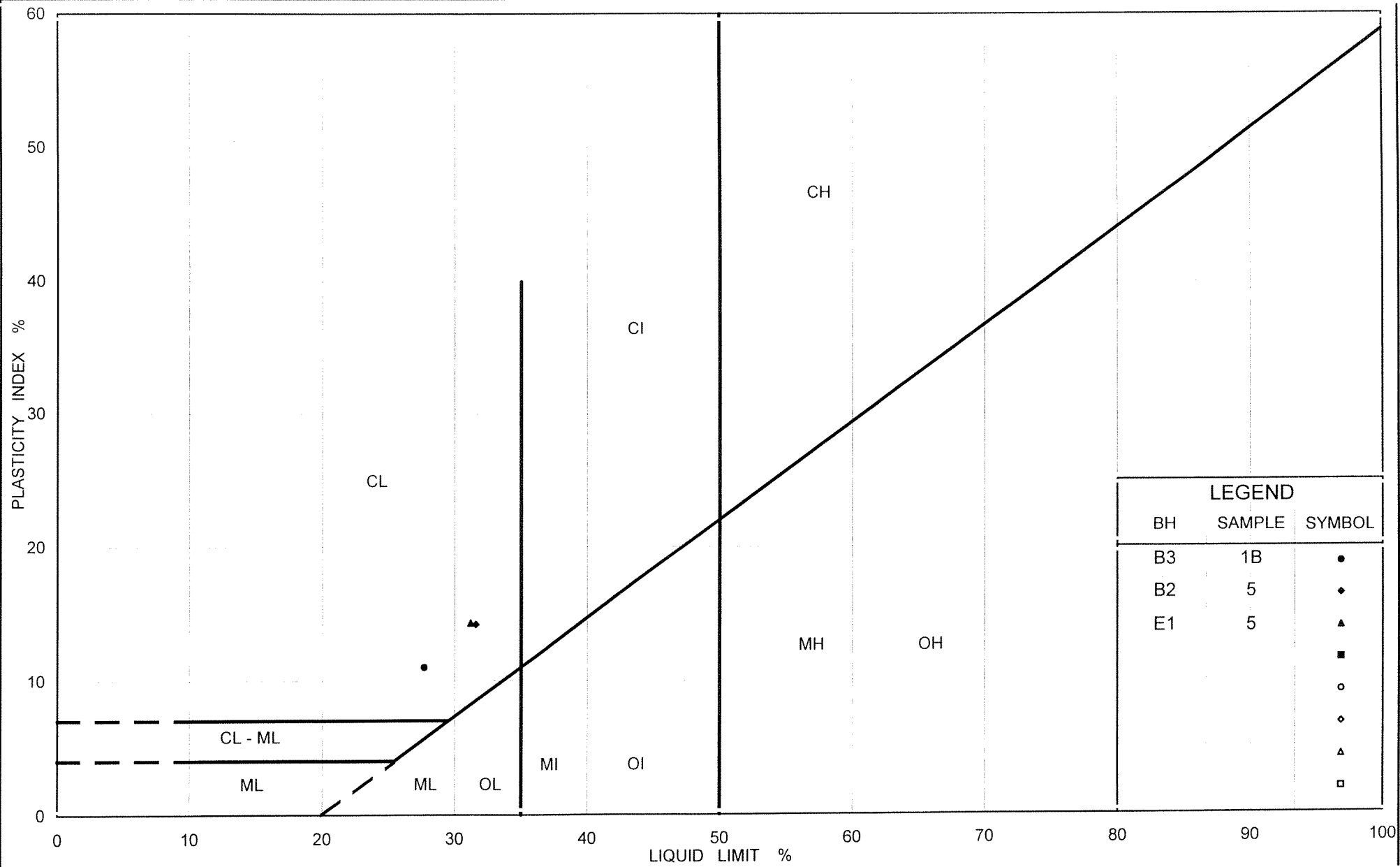
Embankment Fill

FIGURE 1



LEGEND

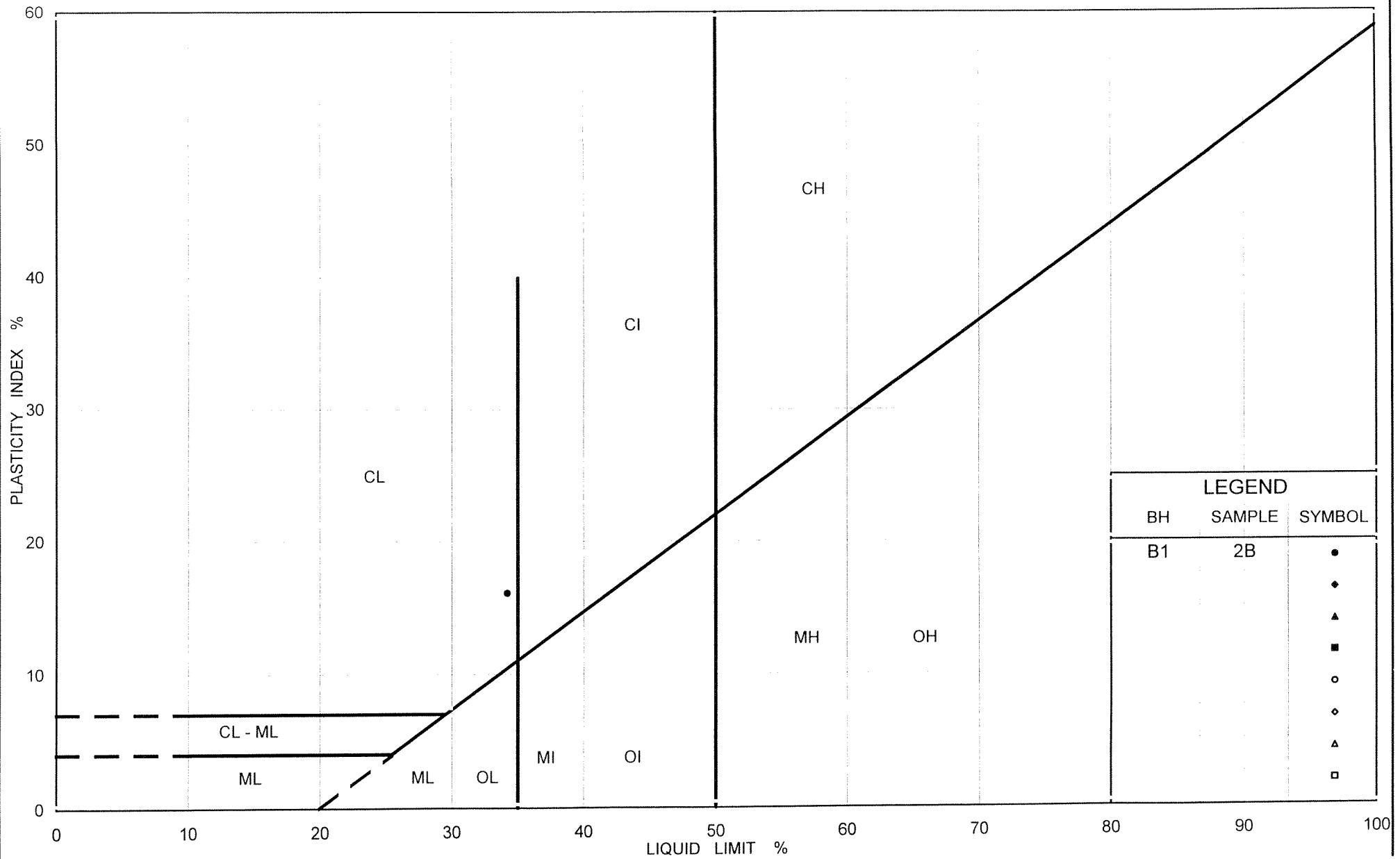
SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B2	5	185.4
■	B2	7	183.1
◆	E1	5	181.0



Ministry of Transportation

PLASTICITY CHART Embankment Fill

FIG No.2
Project No. 001-1141F



Ministry of Transportation

Ontario

PLASTICITY CHART Surficial Clayey Silt

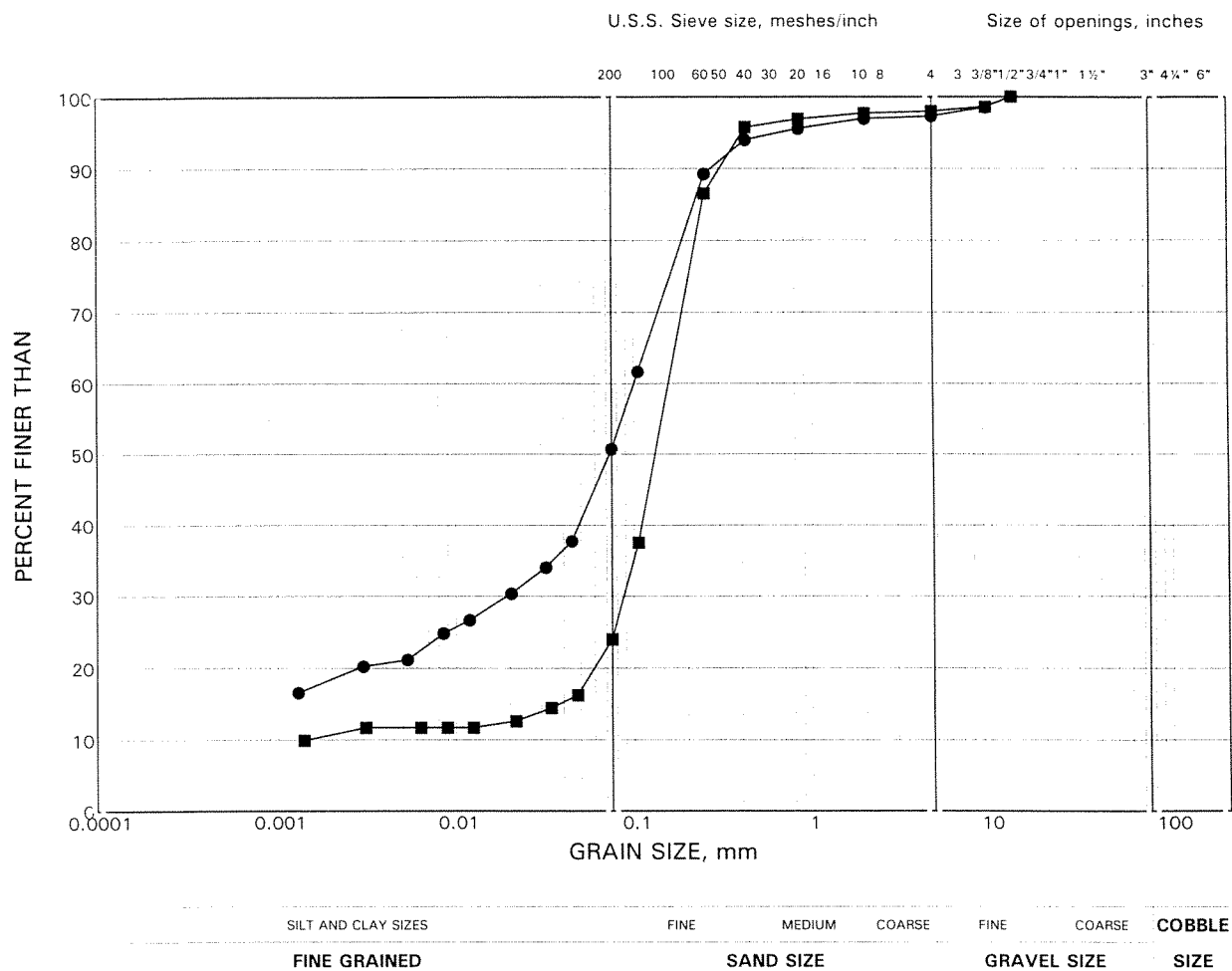
FIG No.3

Project No. 001-1141F

GRAIN SIZE DISTRIBUTION TEST RESULTS

Surficial Sand and Silt to Silty Sand

FIGURE 4



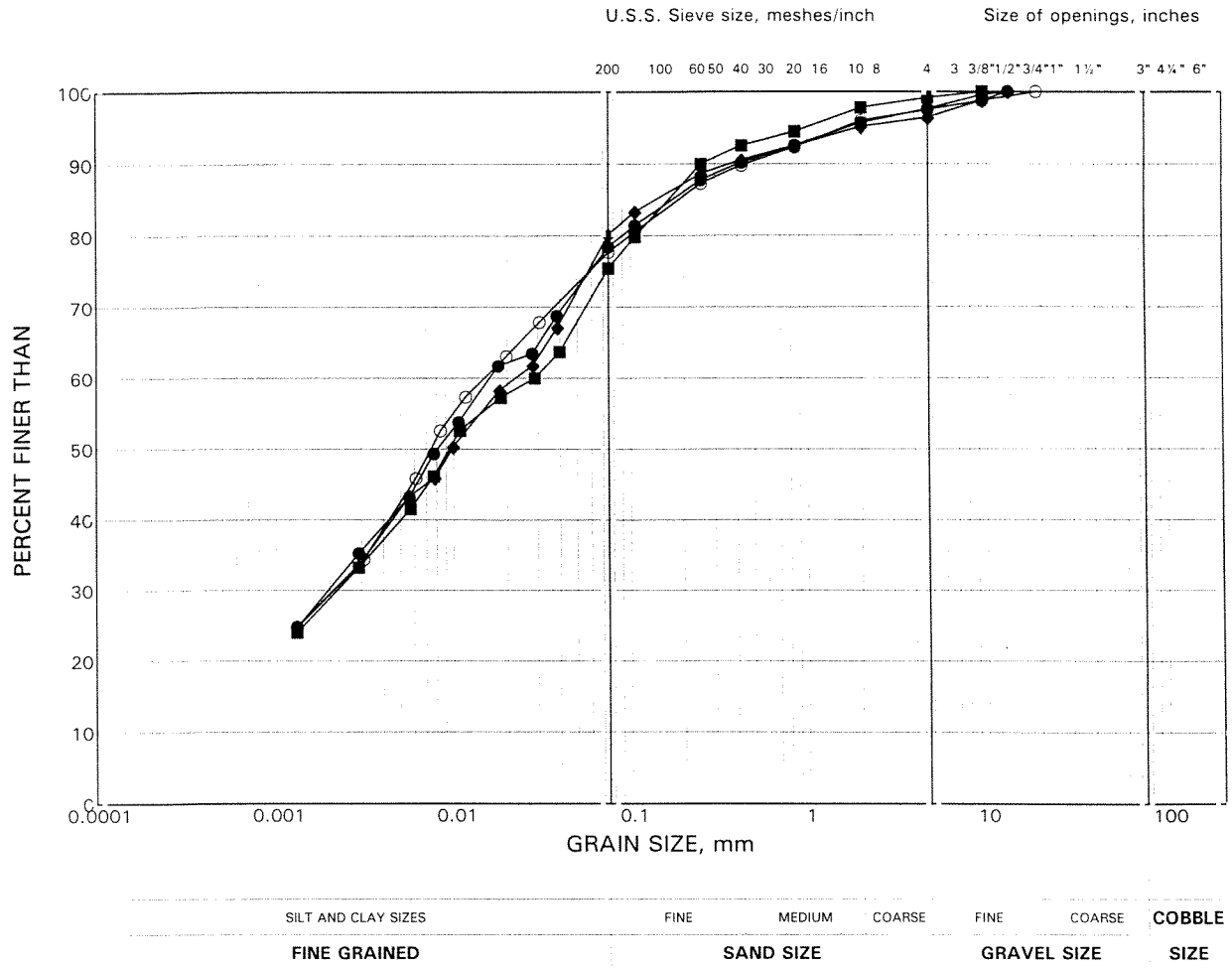
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1	3A	178.2
■	B3	3	184.2

GRAIN SIZE DISTRIBUTION TEST RESULTS

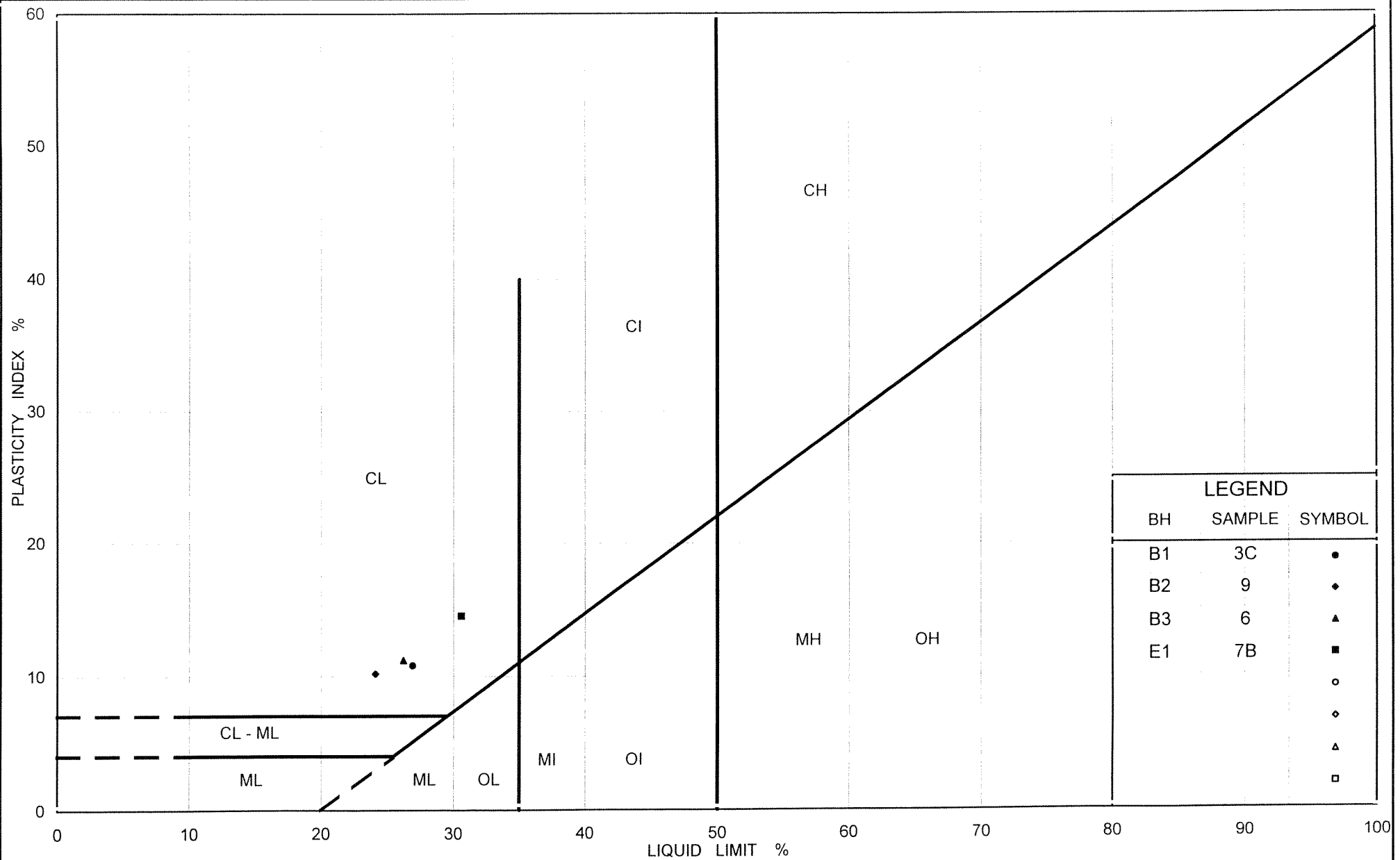
Clayey Silt Till

FIGURE 5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
●	B1	3C	177.5
■	B2	9	179.8
◆	B3	6	182.0
○	E1	7B	177.8



Ontario

Ministry of Transportation

PLASTICITY CHART Clayey Silt Till

FIG No.6

Project No. 001-1141F