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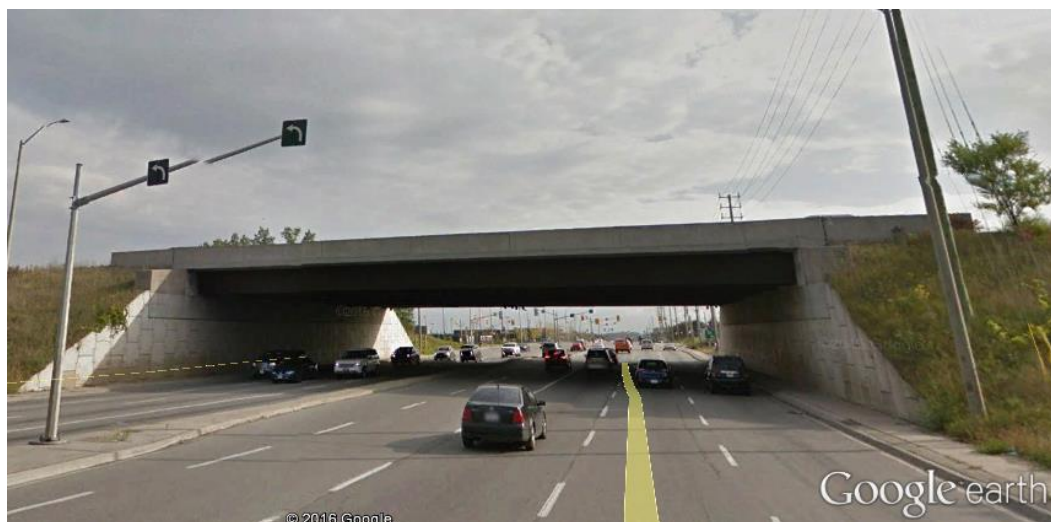
DRAFT PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

MAPLEVIEW DRIVE OVERPASS, SITE NO. 30-179
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
W.O. 06-20016

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DRAFT REPORT



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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
MAPLEVIEW DRIVE OVERPASS – SITE NO. 30-179
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the preliminary design for the widening of the Maplevue Drive Overpass in the City of Barrie. The proposed work is part of the preliminary and design-build ready design associated with the Highway 400 widening from 1 km south of Highway 89 to the junction of Highway 11 in Simcoe County, Ontario.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated July 2013. Golder's scope of work for foundation engineering services associated with the Maplevue Drive rehabilitation is contained in Section 5.8 of AECOM's (previously URS Canada) Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for foundation engineering services for this project, dated January 20, 2014.

This report addresses the proposed widening of the Maplevue Drive Overpass and the associated approach embankments and cuts (MTO Structure Site No. 30-179). The work for this structure site is being administrated as a preliminary design assignment. The Preliminary Foundation Investigation Report is intended for planning and preliminary purposes only and the Contractor shall satisfy himself as to the sufficiency of the subsurface information and supplement the information as needed to meet the requirements for detail design.

2.0 SITE DESCRIPTION

The Maplevue Drive Overpass, which is part of the Highway 400-Maplevue Drive (formerly Molson Park Drive) Interchange, is located approximately 3.2 km south of the Essa Road Interchange, in the City of Barrie. The existing Maplevue Drive Overpass is an about 49 m wide by 39 m long single-span structure with integral abutments supported on driven steel H-piles. An approximately 13.4 m wide detour structure parallels the west side of the southbound lanes (SBL).

The overall surface topography in the vicinity of the site is relatively flat and consists of commercial developments and an industrial area to the west and east of Highway 400, respectively. The natural ground surface at the site ranges between approximately Elevations 301 m and 299 m. At this structure site, Highway 400 has been constructed on an up to 5.5 m high embankment and has an existing grade of about Elevation 304.5 m to 304 m. Maplevue Drive was constructed in a cut with an existing road grade of about Elevation 297 m.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Borehole Investigation

Nine boreholes were advanced at this site as part of a MTO geotechnical investigation in 1997 (MTO, 1997 and 2002) for the replacement of the existing Maplevue Drive Overpass structure, associated with the widening of Highway 400. Boreholes 1, 2, 5 and 6 were advanced at the proposed north abutment and Boreholes 3, 4, and 7 to 9 were advanced at the proposed south abutment, to depths between 18.7 m and 27.7 m below ground surface. Additionally, Direct Cone Penetration Tests (DCPT) were advanced directly adjacent to each borehole, and met refusal at much shallower depths than the boreholes were terminated at. The borehole locations are shown on Drawing 1.



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The boreholes were advanced using 82 mm inside diameter (I.D.) continuous flight hollow stem augers and soil samples were obtained at intervals of depth of about 0.75 m, 1.5 m, and 3.0 m using a 50 mm outer diameter split-spoon sampler driven by a manual hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586).

The water level in the open boreholes was observed during and following the drilling operations. Laboratory index and classification testing were carried out on select soil samples.

The borehole locations in MTM NAD83 northing and easting coordinates, ground surface elevations reference to Geodetic datum and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
1	4,910,629.3	290,009.5	300.9	24.8
2	4,910,614.6	290,013.0	296.1	18.7
3	4,910,591.1	290,015.3	296.1	21.8
4	4,910,569.8	290,021.0	300.4	27.7
5	4,910,630.5	290,044.8	301.8	24.0
6	4,910,650.4	290,078.7	302.0	24.2
7	4,910,593.7	290,050.8	301.3	24.8
8	4,910,608.4	290,088.2	301.3	18.7
9	4,910,586.7	290,097.7	300.2	24.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 400 from 6 km south of Highway 89 to the junction of Highway 11 traverses, generally in a south–north direction, the following physiographic regions: the Peterborough Drumlin Field; the Simcoe Lowlands; and the Simcoe Uplands. Along Highway 400, the Peterborough Drumlin Field is present from the southern limit of the project site to south of Line 13 of the Township of Bradford West Gwillimbury, as well as between about 1 km north of Highway 89 to about Essa Road. The Simcoe Lowlands covers the area from south of Line 13 to approximately 1 km north of Highway 89 and from about Essa Road to just north of Anne Street. The Simcoe Uplands extends from just north of Anne Street to beyond the northern limit of this project site.

The surficial soils in the western portion of the Peterborough Drumlin Field, which encompasses the Mapleview Drive site, consist primarily of sandy till deposits and sand to sand and gravel deposits. Deposits of silt, clay or peat may also be found in the low-lying areas between drumlins and eskers.

Along Highway 400, the Simcoe Lowlands include: the Holland River valley; the lowlands of the Lake Simcoe basin to the east; the lowlands of the Nottawasaga basin to the west, which includes Innisfil Creek and the Nottawasaga River to the south and west of the project limits, respectively. The Lake Simcoe and Nottawasaga

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



basins are connected by a flat floored valley through Barrie which extends from the shores of Kempenfelt Bay west generally along Highway 90. The Simcoe Lowlands are generally characterized by deep deposits of deltaic or lacustrine silts, sands and clays associated with glacial Lake Algonquin.

The Simcoe Uplands consist of till plains and ancient shorelines. The till deposits range from clayey to silty and generally become more sandy and containing more boulders in the north. The low-lying areas of this region may also contain shallow deposits of sand and gravel associated with former glacial lake shorelines.

4.2 Subsurface Conditions

The Record of Borehole sheets and laboratory testing results from the MTO 1997 investigation are presented in Appendix A. The interpreted stratigraphic profile and cross-sections are shown on Drawings 1 and 2.

The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the previous investigation as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. According to the Canadian Foundation Engineering Manual (*CFEM*, 2006), the energy delivered to the drill rod varies with the hammer release system, hammer type, anvil and operator characteristics.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of a layer of topsoil and/or fill underlain by a silty sand deposit, which is underlain by a silt deposit, which is underlain by a clayey silt to silty clay deposit, which is in turn underlain by a silty sand to sand to gravelly sand deposit which extends to the refusal condition.

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

4.2.1 Topsoil

Topsoil was encountered at ground surface in Boreholes 1, 4 and 6.

4.2.2 Silty Sand Fill

A 1.0 m to 1.9 m thick fill deposit comprised of silty sand was encountered at ground surface in Boreholes 5 and 7 to 9 and below the topsoil in Borehole 6. Organic material was encountered at the base or below the fill layer in Boreholes 5, 7 and 8.

The SPT 'N'-values measured within the fill deposit range from 7 blows to 27 blows per 0.3 m of penetration, indicating a loose to compact relative density.



4.2.3 Silty Sand

A 0.8 m to 6.9 m thick deposit of silty sand was encountered between Elevations 300.9 m and 296.1 m in all boreholes advanced on site. The deposit was encountered at ground surface in Boreholes 2 and 3, below the topsoil in Boreholes 1 and 4 and below the fill in Boreholes 5 to 9. Additionally, refusal to further penetration was encountered in this deposit in DCPT's drilled directly adjacent to Boreholes 3 to 9.

The SPT 'N'-values measured within the silty sand deposit generally range from 21 blows to 269 blows per 0.3 m of penetration, indicating a compact to very dense relative density. The SPT 'N'-value measured near the top of the deposit, below the fill deposit range from 3 blows to 14 blows per 0.3 m of penetration, indicating a loose to compact relative density.

The resulting envelope of grain size distribution tests completed on samples of the silty sand deposit are shown on Figure 1 in Appendix A.

4.2.4 Silt

A 1.5 m to 3.4 m thick deposit of silt was encountered below the silty sand deposit between Elevation 299.4 m and 239.0 m in all boreholes advanced on site. A hard, clayey silt crust was encountered at the top of this deposit in Borehole 3.

The SPT 'N'-values measured within the silt deposit generally range from 23 blows to 63 blows per 0.3 m of penetration, indicating a compact to very dense relative density. The SPT 'N'-values of the silt deposit encountered near the surface, in Borehole 1, measured 9 blows and 11 blows per 0.3 m penetration indicating a loose to compact relative density, whereas in Borehole 6, SPT "N"-values of 103 blows per 0.3 m of penetration and 162 blows per 0.25 m of penetration were measured. The SPT 'N'-value measured in the clayey silt crust in Borehole 3 is 43 blows per 0.3 m of penetration, suggesting a hard consistency.

The natural water content measured on a sample of the clayey silt crust is about 18 per cent.

The resulting envelope of the grain size distribution tests completed on samples of the silt deposit are shown on Figure 2 in Appendix A. One grain size distribution test completed on the clayey silt crust indicates a gradation comprised of 5 per cent sand and 95 per cent fines.

An Atterberg limits test carried out a sample of the clayey silt crust measured a liquid limit of about 26 per cent, a plastic limit of about 15 per cent, corresponding to a plasticity index of about 11 per cent, indication that the crust is comprised of clayey silt of low plasticity. The results of the Atterberg limits test is shown on Figure 4 in Appendix A.

4.2.5 Clayey Silt to Silty Clay

A 0.9 m to 2.5 m thick deposit of clayey silt to silty clay was encountered between Elevations 296.2 m and 290.1 m below the silt deposit in all boreholes advanced at the site.

The SPT 'N'-values measured within the clayey silt to silty clay deposit generally range from 13 blows to 69 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. A SPT 'N'-value of 133 blows per 0.23 m of penetration was measured at the interface between the cohesive deposit and underlying non-cohesive deposit in Borehole 4.



The natural water content measured on samples of the clayey silt to silty clay range from about 18 per cent to 27 per cent.

Atterberg limits test carried out on eleven samples of the clayey silt to silty clay deposit measured liquid limits ranging between about 31 per cent and 47 per cent, plastic limits between about 16 per cent and 21 per cent and plasticity indices between about 14 per cent and 27 per cent, indicating that the cohesive deposit is comprised of clayey silt of low plasticity to silty clay of intermediate plasticity, as shown on Figures 3 and 4 in Appendix A.

4.2.6 Silty Sand to Sand to Gravelly Sand

A silty sand to sand deposit was encountered in all of the boreholes, below the clayey silt to silty clay deposit between Elevations 295.0 m and 288.5 m, and was penetrated for 7.5 m to 18.9 m before termination of boreholes in refusal conditions. Gravelly sand was encountered within the upper 2.5 m of the silty sand to sand deposit in Boreholes 1, 2, 4, 5, 7 and 8. Additionally, refusal to further penetration was encountered in this deposit in DCPT's drilled directly adjacent to Boreholes 1 and 2.

The SPT 'N'-values measured within the silty sand to sand to gravelly sand deposit range from 85 blows per 0.3 m of penetration to 136 blows per 0.05 m of penetration, indicating a very dense relative density.

The resulting envelope of the grain size distribution tests completed on samples of the silty sand to sand component of the deposit and on samples of the gravelly sand component of the deposit are shown on Figures 5 and 6, respectively, in Appendix A.

4.3 Groundwater Conditions

Perched water was encountered at all borehole locations between about Elevation 297.5 m and 294 m, typically above the silt layer within the silty sand deposit (in Boreholes 2 to 4 and 6 to 9), and within the silt deposit (in Boreholes 1 and 5). It is considered that this shallow water table is perched by the clayey silt to silty clay deposit underlying the silty sand and silt deposits. Below the clayey silt to silty clay deposit, the groundwater level upon completion of drilling was measured at a depth of 27.6 m in Borehole 4, corresponding to Elevation 272.8 m. All other boreholes advanced on site were dry upon completion of drilling.

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.



5.0 CLOSURE

This report was prepared by Ms. Madison Kennedy, B.A.Sc., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

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PART B

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
MAPLEVIEW DRIVE OVERPASS – SITE NO. 30-179
HIGHWAY 400 WIDENING
FROM 1 KM SOUTH OF HIGHWAY 89 TO JUNCTION OF HIGHWAY 11
MINISTRY OF TRANSPORTATION, ONTARIO
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6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed widening of the Highway 400-Mapleview Drive Overpass (MTO Structure Site No. 30-179). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 1997 subsurface investigation at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and design will be required during the detail design phase of the project.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required during construction. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

Golder Associates Ltd. (Golder) has been retained by AECOM (formerly URS Canada Inc.) on behalf of the Ministry of Transportation, Ontario (MTO) to provide recommendations on foundation aspect for the preliminary design of the Highway 400-Mapleview Drive Overpass in the City of Barrie. It is understood that the Mapleview Drive Overpass will consist of a single-span, pre-cast girder bridge with an about 39 m span length, about 71 m wide, including the incorporation of the existing detour structure which parallels the west side of the southbound lane (SBL).

Based on the General Arrangement (GA) Drawing provided by AECOM on May 11, 2016, the grade of the proposed Overpass is about Elevations 304.5 m and 304 m at the north and south ends, respectively. In comparison, the proposed grade for Mapleview Drive is about Elevation 297 m.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary, the proposed overpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the year in which the previous geotechnical investigation was carried out, the location so fthe boreholes relative to the locations of the structure foundations and the limited lab testing completed to date, as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a “low degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , and embankment settlement factor, ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.7 below.



6.3 Foundation Options

As part of the future widening of Highway 400 in Simcoe County, the existing Maplevue Drive Overpass will require widening. According to the available information, the existing single-span northbound lane (NBL) and SBL structure has integral abutments supported by H-piles that have a pile cap with its underside at Elevations 300 m at the south abutment and 300.5 m at the north abutment, and pile tips at approximately Elevation 278 m, as outlined in the construction drawings contained in the 1997 Foundation Investigation and Design Report (MTO, 1997). The proposed rehabilitation/widening of Highway 400 consists of an approximately 4.3 m widening to the east and west of the proposed centreline. It is understood that a 13.4 m wide detour bridge was constructed in 1998 as part of the replacement of the then existing bridge and that the detour bridge will be incorporated into the future widening. In addition, the north and south abutments of the detour bridge have been constructed on pile foundations to accommodate an additional lane without the need for any additional foundation system. The Highway 400 centreline will be re-positioned to the west of the existing alignment and the Highway grade will be maintained at approximately Elevations 304.5 m and 304 m at the respective ends. There are no changes proposed to the existing Maplevue Drive roadway grade.

It is understood that, at this time, this bridge is to be rehabilitated/widened as part of the widening of Highway 400. Given that the existing bridge abutment are integral abutments, the rehabilitation/widening of the bridge foundations should be completed with integral abutments consistent with those of the existing structure; however, recommendations are provided herein for both shallow foundation and deep foundation options for the support of the abutments and proposed bridge structure, in the event that the bridge requires a full replacement. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the foundation alternative for a new (replacement) structure is presented in Table 1.

- **Shallow foundations – spread/strip footings:** Shallow foundations comprised of spread or strip footings, founded on the compact to very dense silty sand and silt deposits, or constructed on a compacted Granular 'A' pad are feasible for support of the new abutments; however, this foundation type will preclude the use of integral abutments and is not recommended for the rehabilitation/widening of the existing structure, given that it is incompatible with the design of the existing foundations.
- **Deep foundations – driven steel H-piles or pipe (tube) piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for H-piles and pipe piles) or integral abutments (for H-piles only). Given that the existing bridge abutments are integral abutments, driven H-piles are the preferred foundation alternative for a bridge rehabilitation/widening. For widening, the existing structure should be monitored for vibration and displacement (lateral/vertical) during driving of new piles adjacent to the existing piles.
- **Deep foundations – drilled shaft (caissons):** Drilled shafts (caissons) are considered feasible for the support of the abutments for a full bridge replacement; however this option would preclude integral abutment design and is not recommended for the rehabilitation/widening of the structure, given that it is incompatible with the design of the existing foundations. In addition, this option would be more expensive than either shallow foundations or driven pile foundations, although fewer caisson elements would be required in comparison to the number of driven steel piles that would be required if the structure is replaced. If caissons are adopted for support of the abutments, they would extend into and through non-cohesive soil deposits; temporary liners would be required during construction to control potential ground losses and/or disturbance of the caisson base.



Based on the above considerations, pile foundations using driven steel H-piles are preferred from a foundations perspective.

6.4 Shallow Foundations

6.4.1 Founding Elevation

For the support of the abutments of a full replacement structure, spread/strip footings should be founded on the compact to very dense silty sand and silt deposits, or on compacted granular pads. Where spread/strip footings are to be founded on the native soil, the highest founding elevations recommended for preliminary design of footings are presented below.

Foundation Element	Highest Founding Elevation (m) ¹	Founding Soil
North and South Abutment	295.5	Compact to very dense silty sand and silt deposits
Footing on minimum 1 m thick compacted Granular 'A' pad	295.5	Compacted Granular 'A'

It should be noted that the highest founding elevation takes into consideration the proposed Mapleview Drive grade and the depth of frost penetration.

6.4.2 Factored Geotechnical Axial Resistances

The following factored ultimate and serviceability geotechnical resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for preliminary design of spread/strip footing founded on the properly prepared silty sand or silt deposits, or on a compacted Granular 'A' pad having a minimum thickness of 1 m.

Foundation Alternative	Factored Ultimate Geotechnical Axial Resistance ¹ (at ULS) (kPa)	Factored Serviceability Geotechnical Resistance ¹ (at SLS) for 25 mm of Settlement (kPa)
Footing on properly prepared compact to very dense silty sand or silt deposit	550	150
Footing on minimum 1 m thick compacted Granular 'A' pad	650	175

Note: 1. The factored geotechnical resistances given above are estimated for a 3 m wide spread/strip footing.

The preliminary factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *CHBDC (2014)*.



6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The following presents the coefficient of friction, $\tan \phi'$, for the interface between the concrete footing and the silty sand and silt deposits or Granular 'A' pad.

Founding Material	Coefficient of Friction ($\tan \phi'$)
Cast-in-place concrete footing on native compact to very dense silty sand / silt	0.40
Cast-in-place concrete footing on compacted Granular 'A' pad	0.60

6.4.4 Frost Protection

The footings should be provided with a minimum 1.5 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.5 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

6.5.1 Founding Elevation

The abutments for the rehabilitated/replacement structure may be supported on steel H-piles or pipe piles (for a replacement structure) driven to found within the very dense silty sand to sand to gravelly sand deposit.

Based on the GA Drawing, integral abutments have been adopted for the design of the existing structure, and the abutments will be "perched" within the embankments, with the underside of the pile caps at approximately Elevations 300.5 m and 300 m at the north and south abutments, respectively. The following pile tip elevations are recommended for preliminary design, which is consistent with the design tip elevation of the existing bridge.

Foundation Element	Approximate Lowest Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
North Abutment	295.0 to 288.5	278.0	Very dense silty sand to sand to gravelly sand
South Abutment	295.0 to 290.0	278.0	Very dense silty sand to sand to gravelly sand

Based on the above elevations, the proposed piles are estimated to be approximately 22.5 m and 22 m long at the north and south abutments, respectively.



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6.5.2 Factored Geotechnical Axial Resistances

The factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) for driven steel H-piles and closed-end, concrete-filled 324 mm (12-¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) are presented below.

Pile Type	Approximate Length of Driven Pile (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹ (kN)
HP 310x110	18	1,450	N/A
324 mm OD Pipe Pile	18	1,250	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

Pile installation should be in accordance with OPSS 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity.

The preliminary factored geotechnical resistances provided above will have to be re-evaluated and modified, as necessary, during detail design in consideration of additional subsurface investigation at the foundation elements.

6.6 Drilled Shaft (Caisson) Foundations

6.6.1 Founding Elevations

Drilled shafts (caissons) founded within the very dense silty sand to sand to gravelly sand deposit may be considered for support of the abutments for a proposed full replacement structure. The following drilled shaft founding elevations may be used for preliminary design purposes, assuming a minimum of 1 m penetration into "100-blow" soil:

Foundation Element	Approximate Lowest Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
North and South Abutment	295.0 to 288.5	287.0	Very dense silty sand to sand to gravelly sand
North and South Abutment	295.0 to 290.0	287.0	Very dense silty sand to sand to gravelly sand

If drilled shaft foundations are adopted, a temporary liner should be utilized to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.



6.6.2 Geotechnical Axial Resistance/Reaction

The following factored ultimate and serviceability geotechnical axial resistances (at ULS and SLS for 25 mm of settlement, respectively) may be used for design of drilled shaft (caisson) foundations:

Drilled Shaft Diameter (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)	Factored Serviceability Geotechnical Resistance (at SLS) for 25 mm of Settlement ¹ (kN)
0.9	2,400	N/A

Note: 1. The factored serviceability geotechnical resistance (at SLS) for 25 mm of settlement will be greater than the factored ultimate geotechnical axial resistance (at ULS) and as such, the SLS condition does not apply.

The preliminary factored geotechnical resistances provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of any additional subsurface investigation at the foundation elements.

6.7 Approach Embankments

6.7.1 Subgrade Preparation and Embankment Construction

Based on the existing topographic information, the existing Highway 400 embankment side slopes at the Mapleview Drive approach embankment are inclined at about 2 horizontal to 1 vertical (2H:1V). The side slope for the proposed Highway 400 embankment widening should be constructed at a maximum inclination of 2H:1V. Where widening of the existing embankment occurs, benching the existing embankment side slopes to integrate with the new fill should be carried out in accordance with OPSD 208.010 (Benching of Earth Slopes).

6.7.2 Approach Embankment Stability and Factored Settlement

Given the acceptable/satisfactory performance of the existing embankments and that the widening is to be limited to approximately 4.3 m, stability issues are not anticipated within the limits of the approach embankments widening. The factored settlement associated with the widening of the up to 5.5 m high approach embankments to the west of Highway 400 is estimated to be less than 25 mm. Given the generally non-cohesive nature of the overburden, it is anticipated that the majority of the settlement will occur during and shortly after construction of the approach embankments.

6.8 Construction Considerations

The following sections identify future construction considerations that may impact the future design and construction.

6.8.1 Removal of Existing Foundations

If spread/strip footings are adopted for a full bridge replacement, extraction of the existing piles will not be required. However, to reduce potential stress concentrations immediately below the footings, it is recommended that



the existing piles be removed to at least 1 m below the footing founding elevation and the excavation surrounding the piles be backfilled with a compacted Granular 'A' pad.

If drilled shafts (caissons) are adopted for a full bridge replacement, the existing piles should not be extracted from the ground to avoid disturbance of the silty sand to gravelly sand deposit. In addition, to avoid potential conflicts between the existing piles and the drilled shafts during construction, the new abutments should be offset some distance away from the existing abutment.

The required depth of pile removal below the footing founding elevation and/or offset distance between the existing piles and the proposed drilled shafts (caissons) should be determined during detail design.

6.8.2 Open-Cut Excavation

The construction of new spread/strip footings and/or pile caps will require excavations up to about 1.5 m below the existing Mapleview Drive grade and will generally be made through the native compact to very dense silty sand and silt deposits. The native non-cohesive deposits are classified as Type 3 soils, according to the Occupational Health and Safety Act (OHSA) and, as such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V.

6.8.3 Temporary Protection Systems

If a full bridge replacement is considered for the existing bridge, temporary protection systems will be required to facilitate the removal of the existing bridge foundations and construction of the new abutments. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation.

6.8.4 Control of Groundwater

Given the presence of a perched water table above the cohesive deposit, where spread/strip footings or pile caps are to be founded in the native soils it is anticipated that an active dewatering system will be required. Further investigation is required to determine the dewatering requirements due to the perched aquifer. At this preliminary stage it is anticipated that an active dewatering system consisting of pumping from sumps within the excavation would be adequate to maintain a dry excavation.

6.8.5 Vibration Monitoring During Pile Installation

If piles foundations are adopted for the bridge rehabilitation/widening, it is recommended that vibration monitoring be carried out during pile installation. For preliminary design purposes, a maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good conditions.



6.8.6 Monitoring of Existing Structure

It is recommended that the abutment of the existing structure be monitoring for settlement and lateral movement during construction, especially during installation of temporary roadway protection, excavation for the new abutments and during pile driving (where applicable). The foundation monitoring should be carried out by a qualified foundations consultant reporting to the Contract Administrator.

6.8.7 Ground and Groundwater Control for Drilled Shaft (Caisson) Construction

As discussed in Section 6.6, running or flowing of silty sand to gravelly sand deposit could occur during or after drilling of the drilled shaft (caisson), and basal heave could occur at the caisson base. If drilled shaft foundations are adopted, temporary liners would be required to support the overburden soils and balance groundwater pressures during construction. In addition, placement of concrete by tremie methods would be required.

6.8.8 Protection of Subgrade

The non-cohesive soils that will be exposed within the excavations are expected to be below the perched water table at the abutments and will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

6.9 Recommendations for Future Work During Detail Design

During detail design, it is recommended that additional site investigation be carried out to confirm the subsurface conditions for the bridge rehabilitation/widening or replacement, as well as to install piezometers to aid the assessment of the natural and “perched” groundwater conditions.

In addition, it should be noted that the 1997 investigation was carried out using manual hammers and as such, the N-values as presented on the Record of Boreholes are anticipated to be higher than those that would be obtained if automatic hammers were used. As a result, the factored geotechnical resistances and the practical limits for which driven piles can penetrate into the silty sand to sand to gravelly sand deposit will need to be reassessed during detail design.



7.0 CLOSURE

This report was prepared by Ms. Madison Kennedy, B.A.Sc., a member of the geotechnical engineering group, and was reviewed by Mr. Christopher Ng, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M. A. Costa, P.Eng., a Senior Consultant with Golder and Designated MTO Foundations Contact, conducted an independent quality control review of this report.

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MCK/CN/JMAC/mck

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DRAFT PRELIMINARY FOUNDATION REPORT - MAPLEVIEW DRIVE OVERPASS

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Canadian Standards Association (CSA), 2014. *Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6-14*. CSA Special Publication, S6.1-14.

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Ministry of Transportation, Ontario. 2002. *Preliminary Foundation Investigation and Design Report; Molson Park Drive Overpass, Structure Site 30-179; Highway 400 Widening from 1 km South of Highway 89 to Highway 11, G.W.P. 30-95-00*, GEOCRE No. 31D00-471, prepared by Golder Associates Ltd.

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS 903 Construction Specification for Deep Foundations

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario



TABLES



DRAFT PRELIMINARY FOUNDATION REPORT - MAPLEVIEW DRIVE OVERPASS

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> • Feasible for the support of new abutments for bridge replacement • Not compatible with existing foundation system for support of abutment for bridge rehabilitation/widening. 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques. • Lower cost compared to deep foundations. • Construction of new footings would not interfere with the existing foundation supporting the existing structure. 	<ul style="list-style-type: none"> • Requires larger footing excavation and disposal of a larger volume of soil compared to the excavation for a pile cap. • Does not allow for integral abutment construction. • Dewatering system will be required due to the perched groundwater table • Not compatible with the existing foundations for rehabilitation/widening of the structure 	<ul style="list-style-type: none"> • Estimated cost is approximately \$600/m³ for construction of shallow foundations. 	<ul style="list-style-type: none"> • Subgrade should be protected from frost penetration • Softening / loosening of subgrade due to groundwater would require a concrete to be placed working slab immediately after excavation to design depth, inspection and approval of subgrade.
Steel H-piles or pipe piles	<ul style="list-style-type: none"> • Feasible for the support of new abutments and abutment widening with pile cap “perched” within the approach embankments. 	<ul style="list-style-type: none"> • Conventional construction methods for H-pile or steel pipe pile foundations. • Abutment pile caps would be maintained higher than spread footings, thus reducing or eliminating the depth of excavation and protection system requirements. • Steel H-piles allow for integral abutment configuration; and pipe piles allow for semi-integral abutment configuration. 	<ul style="list-style-type: none"> • Dewatering system may be required for pile caps constructed below the Mapleview Drive grade • Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration. • Driving of piles immediately adjacent to the existing foundation could affect the performance of the existing foundation. 	<ul style="list-style-type: none"> • Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction. 	<ul style="list-style-type: none"> • Driving of piles immediately adjacent to the existing foundation could affect the performance of the existing foundations – close supervision and monitoring of vibrations and settlement should be carried out during pile driving operations.



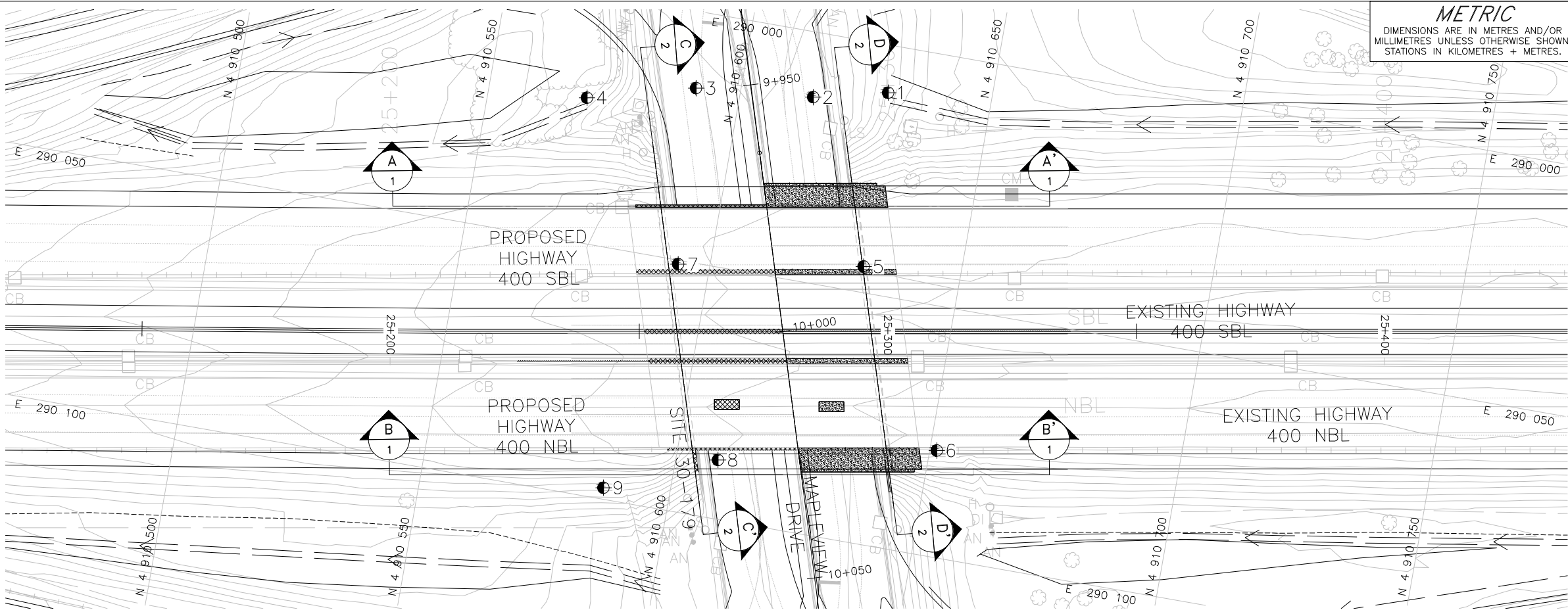
DRAFT PRELIMINARY FOUNDATION REPORT - MAPLEVIEW DRIVE OVERPASS

TABLE 1 – COMPARISON OF REPLACEMENT STRUCTURE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Drilled Shaft (Caissons)	<ul style="list-style-type: none"> • Feasible but not recommended for the support of abutments for bridge replacement • Not compatible with existing foundation system for the support of abutments for bridge rehabilitation/ widening. 	<ul style="list-style-type: none"> • Abutment pile caps could be constructed at the underside of the bridge or maintained higher than spread footings, or H-pile caps, potentially reducing depth of excavation and protection system requirements, or caps can be constructed at level of underside of structure. • Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles. 	<ul style="list-style-type: none"> • Temporary liners would be required; likely not possible to inspect caisson base. • Precludes use of integral abutments. • More expensive compared to shallow foundations. • Not compatible with the existing foundations for rehabilitation/widening of the structure. 	<ul style="list-style-type: none"> • Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for the use of a temporary liner. 	<ul style="list-style-type: none"> • Risk of loosening and leaving in place disturbing founding soils at base of caissons.



DRAWINGS

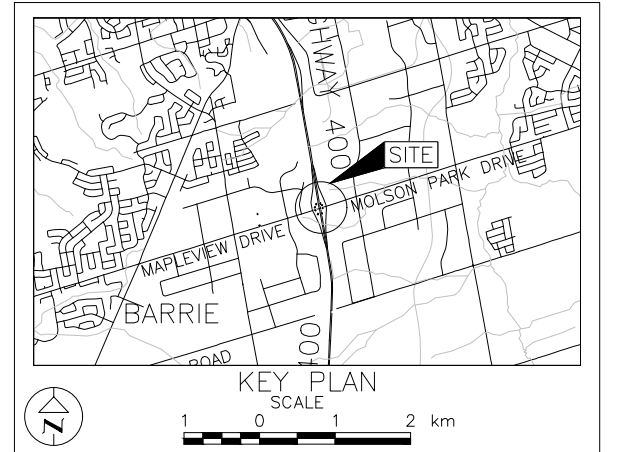


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 06-20016

MAPLEVIEW DRIVE OVERPASS
HIGHWAY 400 WIDENING
BOREHOLE LOCATIONS AND SOIL
STRATA

SHEET



LEGEND

Borehole - Previous Investigation
(Geocress No. 31D00-362)

N

Standard Penetration Test Value

16

Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)

WL upon completion of drilling, or perched

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
1	300.9	4910629.3	290009.5
2	296.1	4910614.6	290013.0
3	296.1	4910591.1	290015.3
4	300.4	4910569.8	290021.0
5	301.8	4910630.5	290044.8
6	302.0	4910651.4	290078.7
7	301.3	4910593.7	290050.8
8	301.3	4910608.4	290088.2
9	300.2	4910586.7	290097.7

NOTES

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

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

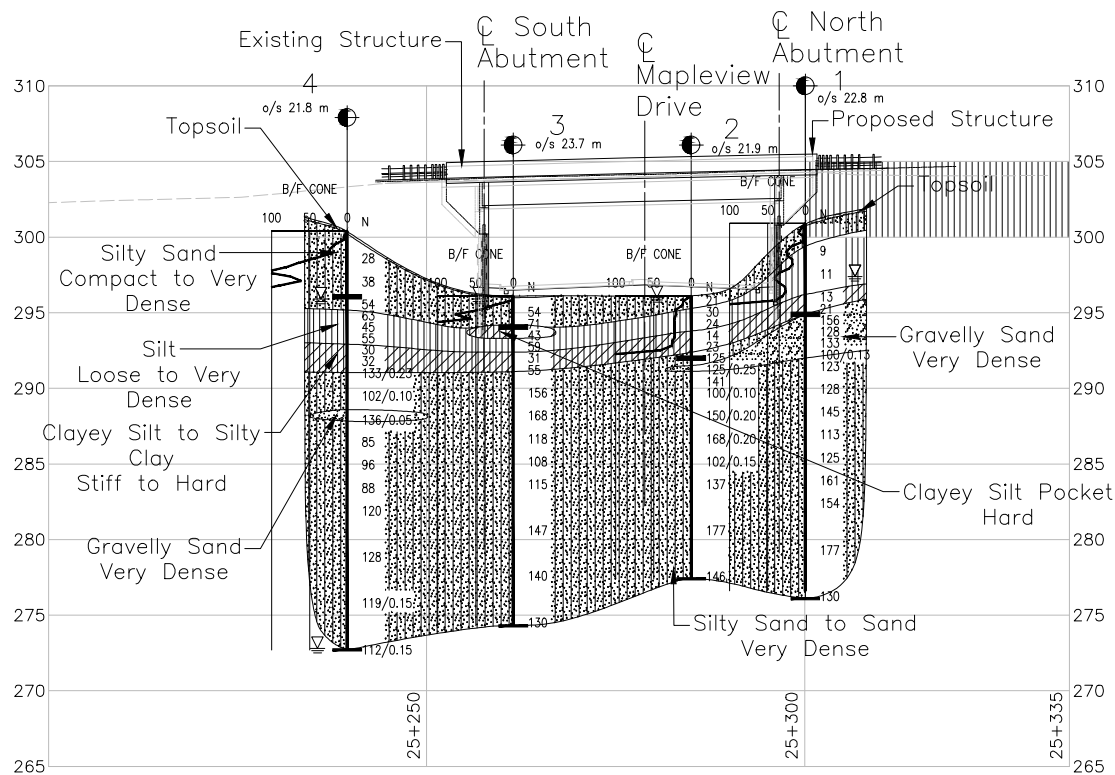
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS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

REFERENCE

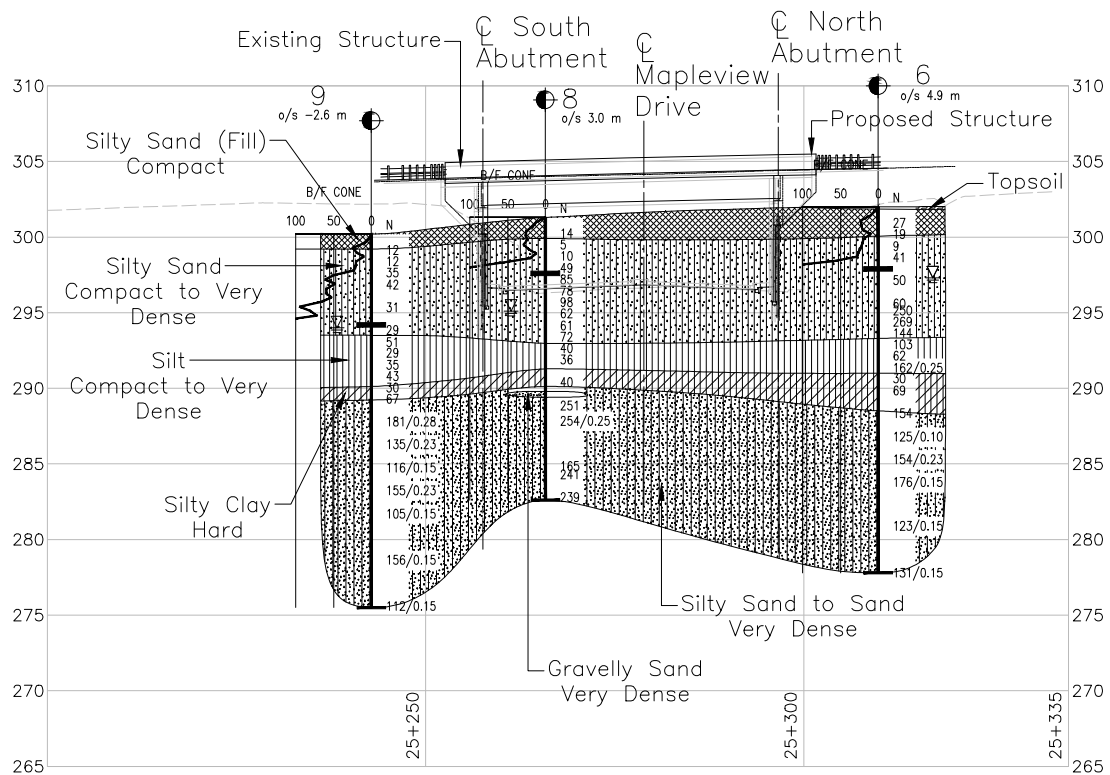
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NO.	DATE	BY	REVISION
Geocres No.,			
HWY. 400		PROJECT NO. 14-1111-0002	DIST. .
SUBM'D. MCK	CHKD. MCK	DATE: 5/25/2016	SITE: 30-179
DRAWN: MR	CHKD. CN	APPD. JMAC	DWG. 1

DRAFT



A-A
1
SBL PROFILE
WEST SIDE OF OVERPASS



B-B
1
NBL PROFILE
EAST SIDE OF OVERPASS

CONT No.
GWP No. 06-20016



MAPLEVIEW DRIVE OVERPASS
HIGHWAY 400 WIDENING
SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- | | |
|---|--|
|  | Borehole — Previous Investigation
(Geocress No. 31D00–362) |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
|  | WL upon completion of drilling, or perched |

BOREHOLE CO--ORDINATES			
No.	ELEVATION	NORTHING	EASTING
1	300.9	4910629.3	290009.5
4	300.4	4910569.8	290021.0
5	301.8	4910630.5	290044.8
6	302.0	4910651.4	290078.7
7	301.3	4910593.7	290050.8
9	300.2	4910586.7	290097.7

NOTES

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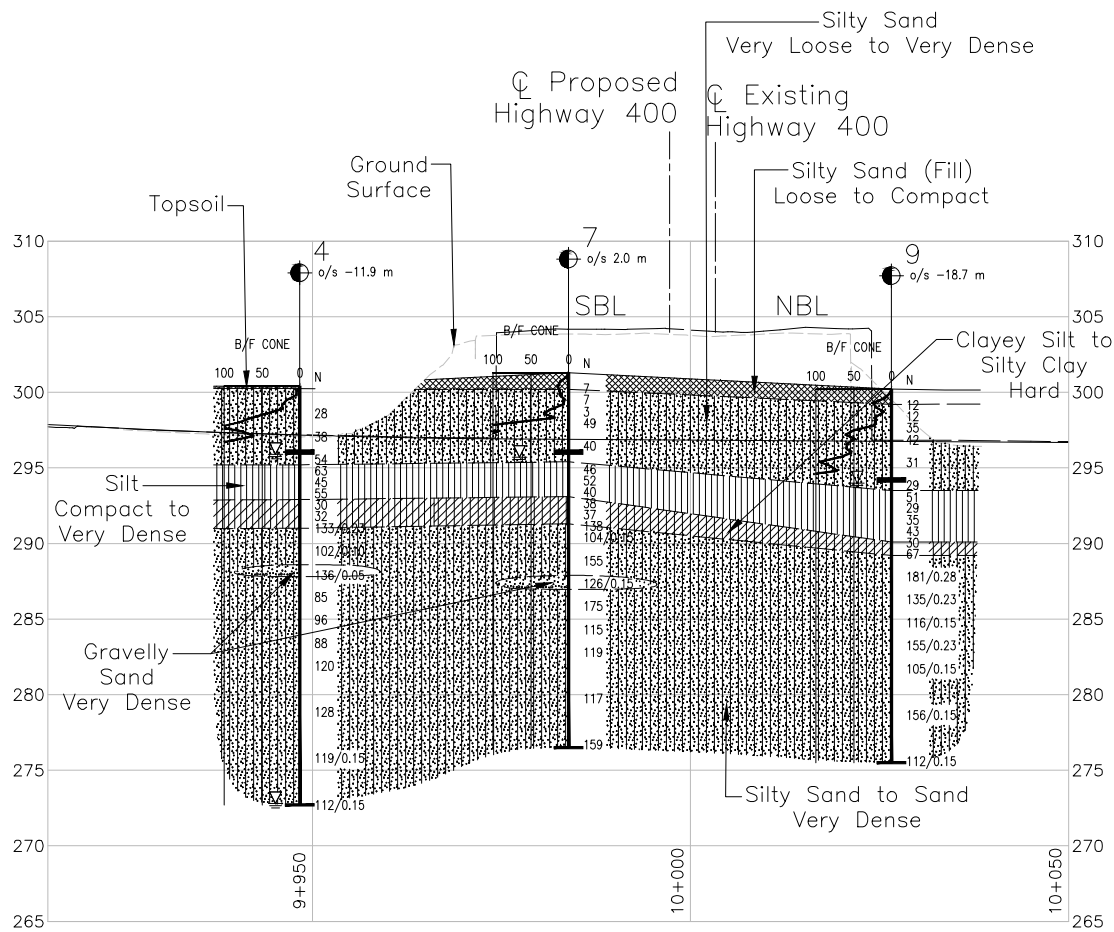
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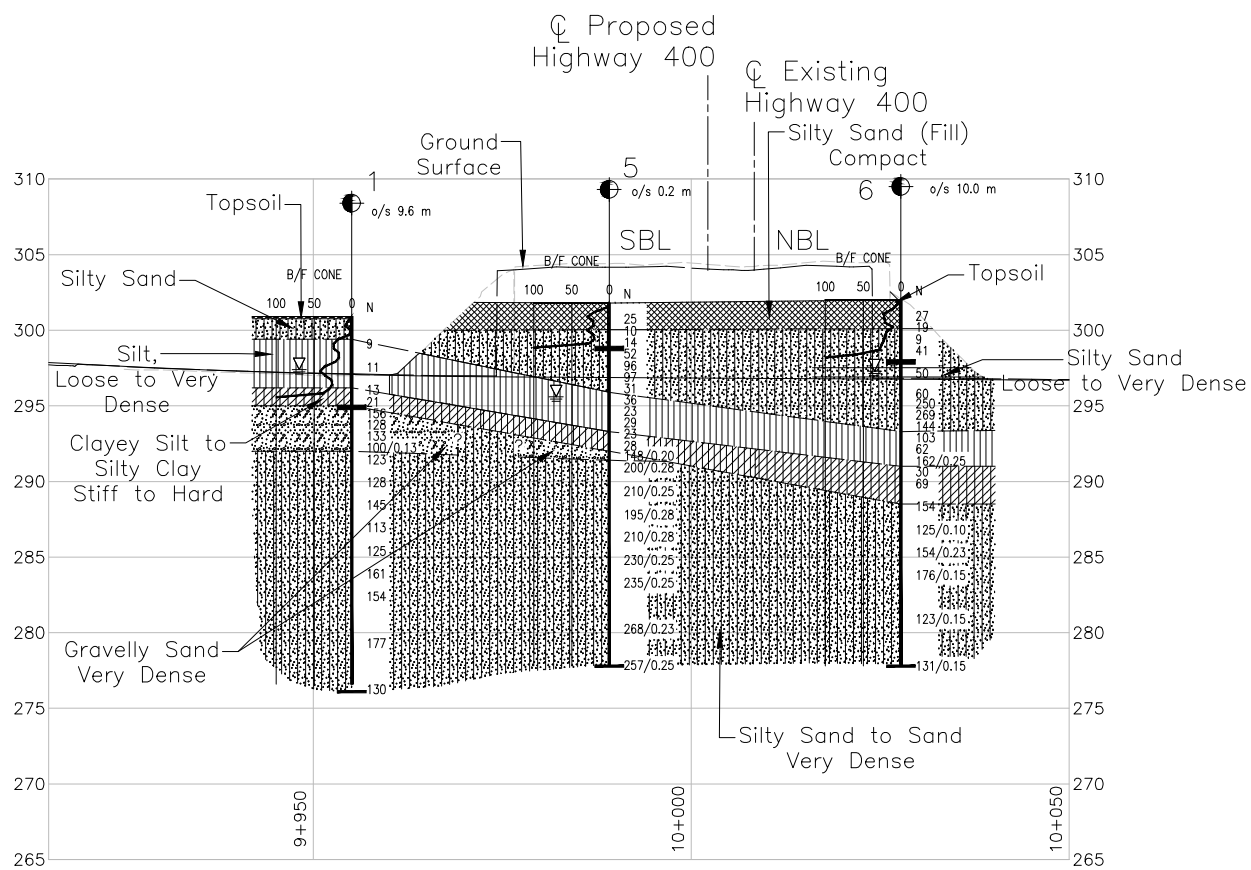
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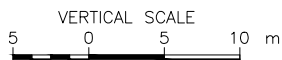
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SUBM'D. MCK		CHKD. MCK	DATE: 5/25/2016
DRAWN: MR		CHKD. CN	SITE: 30-179
		APPD. JMAC	DWG. 2



 SOUTH ABUTMENT AREA
CROSS-SECTION



D-D' NORTH ABUTMENT AREA
1 CROSS-SECTION



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APPENDIX A

**Record of Boreholes and Laboratory Test Results – MTO 1997
Investigation (GEOCRES No. 31D00-362)**

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

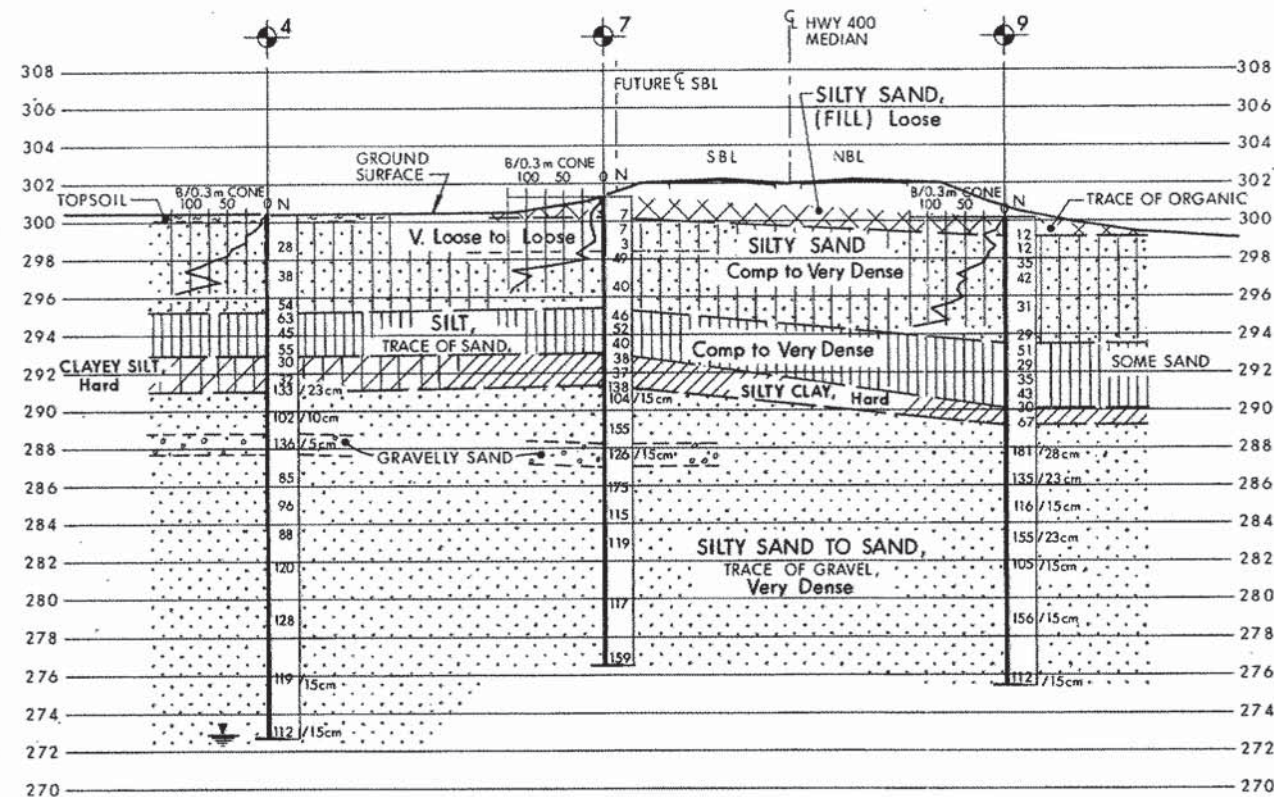
u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

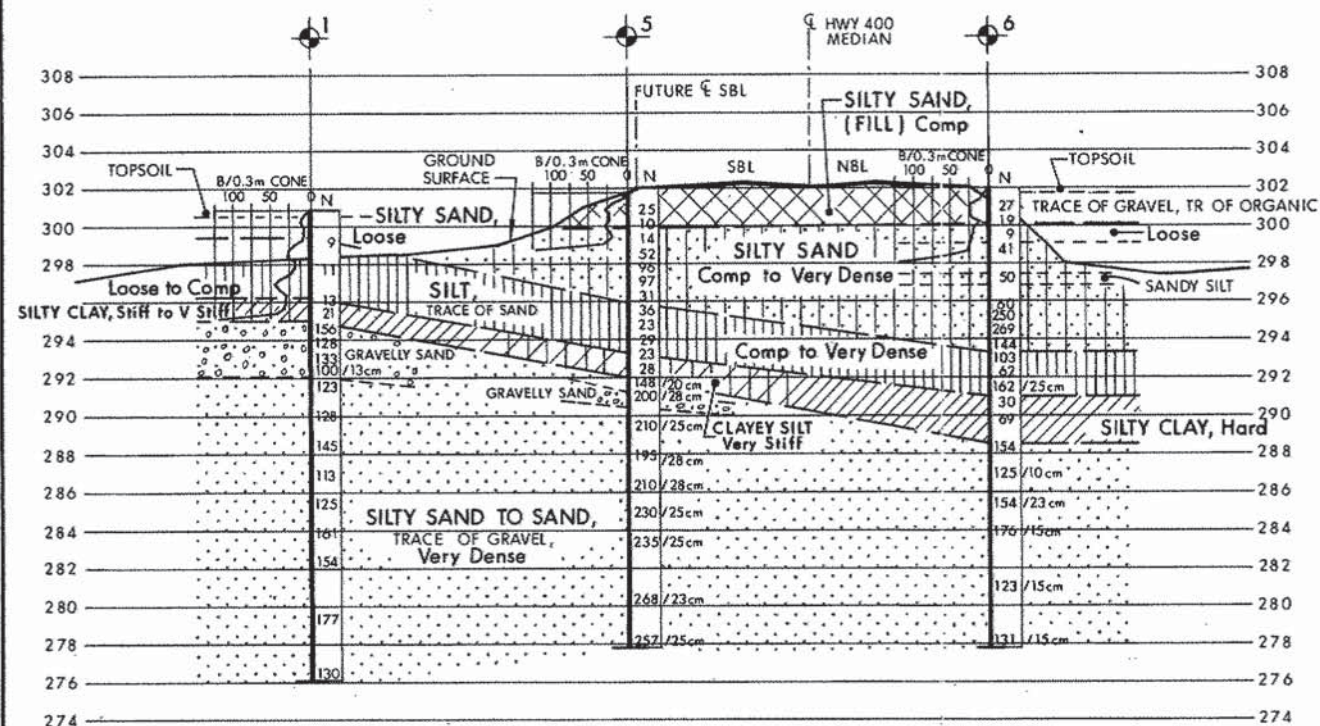
m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

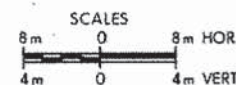
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ²	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



SECTION A-A

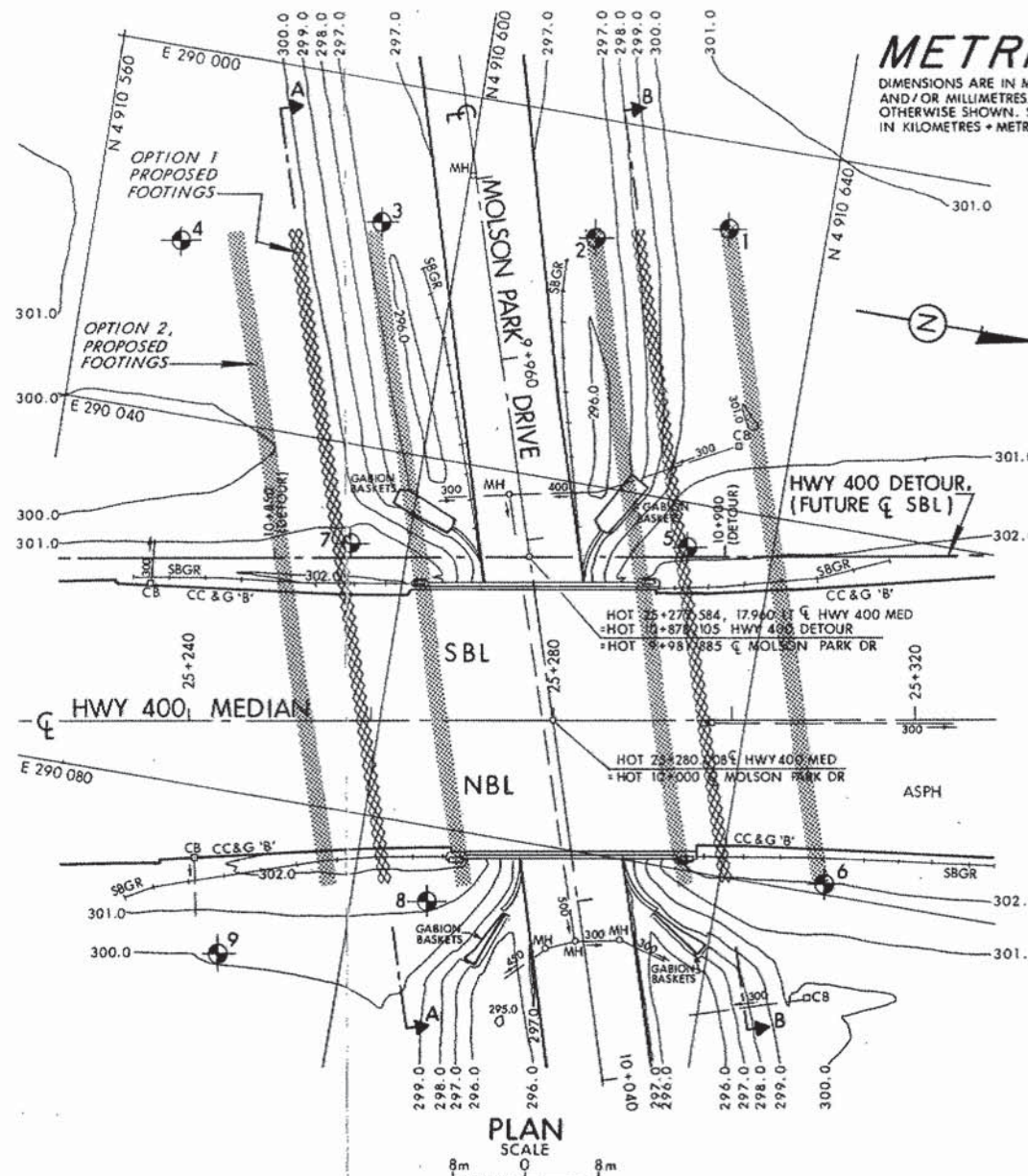


SECTION B-B

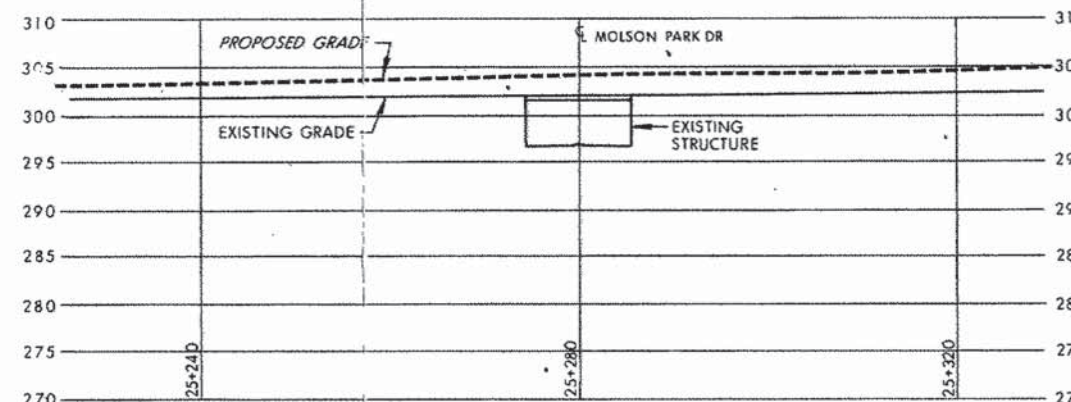


NOTES:

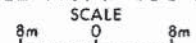
- For Subsoil information of BH's 2, 3 & 8 refer to Record of Borehole Sheets.
- All Boreholes, with the exception of No 4 were dry on completion.
- For Perched Water information refer to Record of Borehole Sheets.



PLAN SCALE 0 8m



PROFILE HWY 400 MEDIAN



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

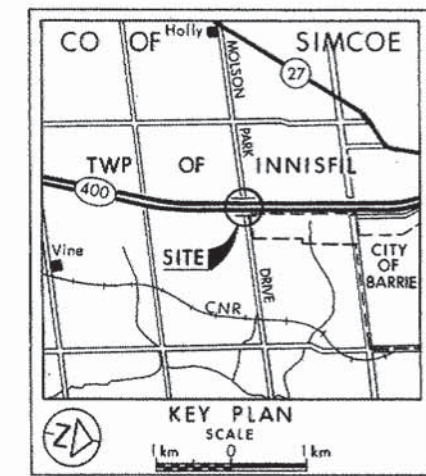
CONT No
WP No 26-96-01

MOLSON PARK DRIVE

BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W at time of investigation 1997 04

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	300.9	4910629.3	290009.5
2	296.1	4910614.6	290013.0
3	296.1	4910591.1	290015.3
4	300.4	4910569.8	290021.0
5	301.8	4910630.5	290044.8
6	302.0	4910651.4	290078.7
7	301.3	4910593.7	290050.8
8	301.3	4910608.4	290088.2
9	300.2	4910586.7	290097.7

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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

REV	DATE	BY	DESCRIPTION

Geocres No 31D-362

HWY No 400	DIST 33
SUBM'D M.V. [CHECKED] DATE 1997 06 12	SITE 30-179
DRAWN R.S. [CHECKED] DATE 1997 06 12	DWG 269601-A



RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 629.3; E 290 009.5 ORIGINATED BY M.V&P C
 DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M.V
 DATUM GEODETIC DATE 1997 04 02 CHECKED BY T.C.K

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _P W W _L	SHEAR STRENGTH kPo ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				
300.9	Ground Surface													
0.0	Topsoil —					*								
299.4	SILTY SAND, Loose													
1.5	SILT, Trace of Sand, Loose to Compact		1	SS	9									0 5 (95)
			2	SS	11									
296.2			3	SS	13									
4.7	SILTY CLAY, Stiff to Very Stiff		4	SS	21									
295.0			5	SS	156									24 66 (10)
5.9	Gravelly Sand <													

RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 25 - 96 - 01 LOCATION CO - ORDS: N 4 910 614.6; E 290 013.0 ORIGINATED BY M.V.
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M.V.
DATUM GEODETIC DATE 1997 04 01 CHECKED BY T.C.K.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	15 30 45		
296.1	Drainage Ditch		1	SS	21								
0.0 295.3	SILTY SAND, Trace of Organic, Compact		2	SS	30								2 18 (80)
0.8	SILT, Some Sand, Compact		3	SS	24								
293.8			4	SS	14								
2.3	SILTY CLAY, Stiff to Very Stiff		5	SS	23								
292.4			6	SS	125								5 81 (14)
3.7			7	SS	125	/25cm							
	Gravelly Sand		8	SS	141	/10cm							15 71 (14)
			9	SS	100								
			10	SS	150	/20cm							
			11	SS	168	/20cm							
			12	SS	102	/15cm							
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		13	SS	137								3 86 (11)
			14	SS	177								
			15	SS	146								
277.4													
18.7	End of Borehole * Note: Borehole Dry on Completion Perched Water Level Encountered at El. 296.0												

RECORD OF BOREHOLE No 3

1 OF 1 METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 591.1; E 290 015.3 ORIGINATED BY M.V.
 DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M.V.
 DATUM GEODETIC DATE 1997 04 14 CHECKED BY T.C.K.

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			20 40 60 80 100	20 40 60 80 100					
296.1	Drainage Ditch													
0.0	Silt and Organic					*								
	SILTY SAND, Very Dense		1	SS	54									0 79 (21)
294.2			2	SS	71									
1.9	Clayey Silt, Hard		3	SS	43		294		120/23cm					0 5 (95)
292.4	SILT, Trace of Sand, Dense to Very Dense		4	SS	59									
3.7	SILTY CLAY, Hard		5	SS	31		292							
291.1			6	SS	55									
5.0														
			7	SS	156		290							6 84 (10)
			8	SS	168		288							10 78 (12)
			9	SS	118		286							
			10	SS	108		284							
			11	SS	115		282							
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		12	SS	147		280							0 94 (6)
							278							
			13	SS	140		276							
274.3			14	SS	130									
21.8	End of Borehole • Note: Borehole Dry on Completion Perched Water Level Encountered at El. 295.9													

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 569.8; E 290 021.0 ORIGINATED BY M V
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V
DATUM GEODETIC DATE 1997 04 10 CHECKED BY T C K

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100					
300.4	Ground Surface													
0.0		Topsoil												
	SILTY SAND, Compact to Very Dense		1	SS	28									
			2	SS	38									
295.2			3	SS	54									0 81 (19)
5.2	SILT, Trace of Sand, Dense to Very Dense		4	SS	63									
			5	SS	45									
292.9			6	SS	55									0 9 (91)
7.5	CLAYEY SILT, Hard		7	SS	30									
			8	SS	32									
291.0			9	SS	133	/23cm								
9.4			10	SS	102	/10cm								
	Gravelly Sand		11	SS	136	/5cm								38 55 (7)
			12	SS	85									2 93 (5)
			13	SS	96									
			14	SS	88									
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		15	SS	120									
			16	SS	128									
			17	SS	119	/15cm								
272.7			18	SS	112	/15cm								1 91 (8)
27.7	End of Borehole													
	* Note: Water Level on Completion at El. 272.8 Perched Water Level Encountered at El. 295.9													

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

METRIC

[illegible]

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 651.4; E 290 078.7 ORIGINATED BY M.V.
 DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M.V.
 DATUM GEODETIC DATE 1997 04 08 CHECKED BY T.C.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							W _P W W _L
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100							
302.0	Ground Surface														
0.0	Topsoil					*									
	SILTY SAND, Trace of Gravel, Trace of Organic, Compact (Fill)		1	SS	27										
300.1			2	SS	19										
1.9			3	SS	9										
	Loose		4	SS	41										
	Sandy Silt		5	SS	50									0 35 (65)	
	SILTY SAND, Dense to Very Dense		6	SS	60										
			7	SS	250										
			8	SS	269										
293.3			9	SS	144									0 71 (29)	
8.7	SILT, Trace of Sand, Very Dense		10	SS	103									0 10 (90)	
			11	SS	62									0 12 (88)	
291.0			12	SS	162	/25cm									
11.0	SILTY CLAY, Hard		13	SS	30										
			14	SS	69										
288.5			15	SS	154									9 78 (13)	
13.5			16	SS	125	/10cm									
			17	SS	154	/23cm									
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		18	SS	176	/15cm									
			19	SS	123	/15cm									
277.8			20	SS	131	/15cm								0 90 (10)	
24.2	End of Borehole														
	* Note: Borehole Dry on Completion Perched Water Level Encountered at El. 297.3														

RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 593.7; E 290 050.8 ORIGINATED BY M V
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V
DATUM GEODETIC DATE 1997 04 11 CHECKED BY T C K

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
301.3	Ground Surface													
0.0	SILTY SAND, Loose													
300.2	(Fill)													
1.1	Organic		1	SS	7									
	Very Loose to Loose		2	SS	7									
			3	SS	3									
			4	SS	49									
	SILTY SAND, Dense		5	SS	40									
295.4														
5.9	SILT, Trace of Sand, Dense to Very Dense		6	SS	46									
			7	SS	52									
293.1			8	SS	40									
8.2	SILTY CLAY, Hard		9	SS	38									
291.3			10	SS	37									
10.0			11	SS	138									
			12	SS	104	/15cm								
			13	SS	155									
	Gravelly Sand		14	SS	126	/15cm								
			15	SS	175									
			16	SS	115									
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		17	SS	119									
			18	SS	117									
			19	SS	159									
276.5														
24.8	End of Borehole													
	Note: Borehole Dry on Completion Perched Water Level Encountered at El. 295.7													

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 608.4; E 290 088.2 ORIGINATED BY P.C.
 DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M.V.
 DATUM GEODETIC DATE 1997 04 04 CHECKED BY T.C.K.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
301.3	Ground Surface													
0.0	SILTY SAND, Compact (Fill)		1	SS	14									
299.9	Organic		2	SS	5									
1.4	Loose		3	SS	10									
			4	SS	49									
			5	SS	85									
	SILTY SAND, Very Dense		6	SS	78									
			7	SS	98									
			8	SS	62									
			9	SS	61									
	Sandy Silt		10	SS	72									
293.0			11	SS	40									
8.3	SILT, Trace of Sand, Dense		12	SS	36									
291.3														
10.0	SILTY CLAY, Hard		13	SS	40									
290.1														
11.2	Gravelly Sand		14	SS	251									
			15	SS	254									
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		16	SS	165									
			17	SS	241									
282.6			18	SS	239									
18.7	End of Borehole + Note: Borehole Dry on Completion Perched Water Level Encountered at El. 295.1													

RECORD OF BOREHOLE No 9

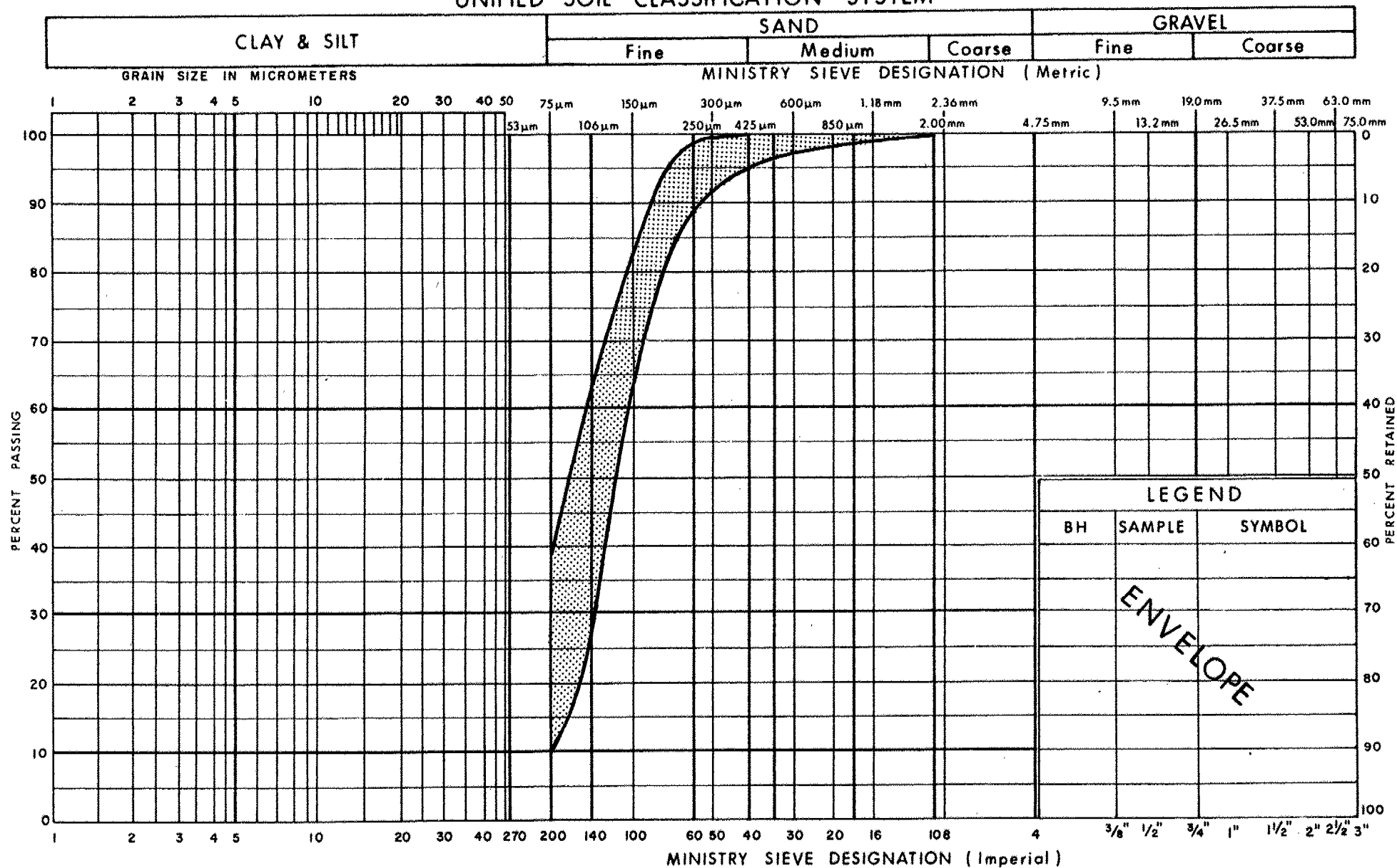
1 OF 1

METRIC

W.P. 26 - 96 - 01 LOCATION CO - ORDS: N 4 910 586.7; E 290 097.7 ORIGINATED BY M V
DIST 33 HWY 400 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V
DATUM GEODETIC DATE 1997 04 07 CHECKED BY T C K

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		FLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							w _p w w _L		
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100							WATER CONTENT (%) 15 30 45		
300.2	Ground Surface																
0.0	SILTY SAND, Trace of Organic, Loose (Fill)					*	300										
299.2			1	SS	12									0 88 (12)			
1.0			2	SS	12												
			3	SS	35												
	SILTY SAND, Compact to Dense		4	SS	42												
			5	SS	31												
293.5			6	SS	29												
6.7			7	SS	51									0 17 (83)			
	SILT, Some Sand, Compact to Dense		8	SS	29												
			9	SS	35												
			10	SS	43									0 24 (76)			
290.1			11	SS	30												
10.1	SILTY CLAY, Hard		12	SS	67												
289.2																	
11.0			13	SS	181	/28cm	288										
			14	SS	135	/23cm	286										
			15	SS	116	/15cm	284										
			16	SS	155	/23cm	282										
	SILTY SAND TO SAND, Trace of Gravel, Very Dense		17	SS	105	/15cm	280							10 79 (11)			
			18	SS	156	/15cm	278										
275.5			19	SS	112	/15cm	276							5 80 (15)			
24.7	End of Borehole * Note: Borehole Dry on Completion Perched Water Level Encountered at El. 294.0																

UNIFIED SOIL CLASSIFICATION SYSTEM



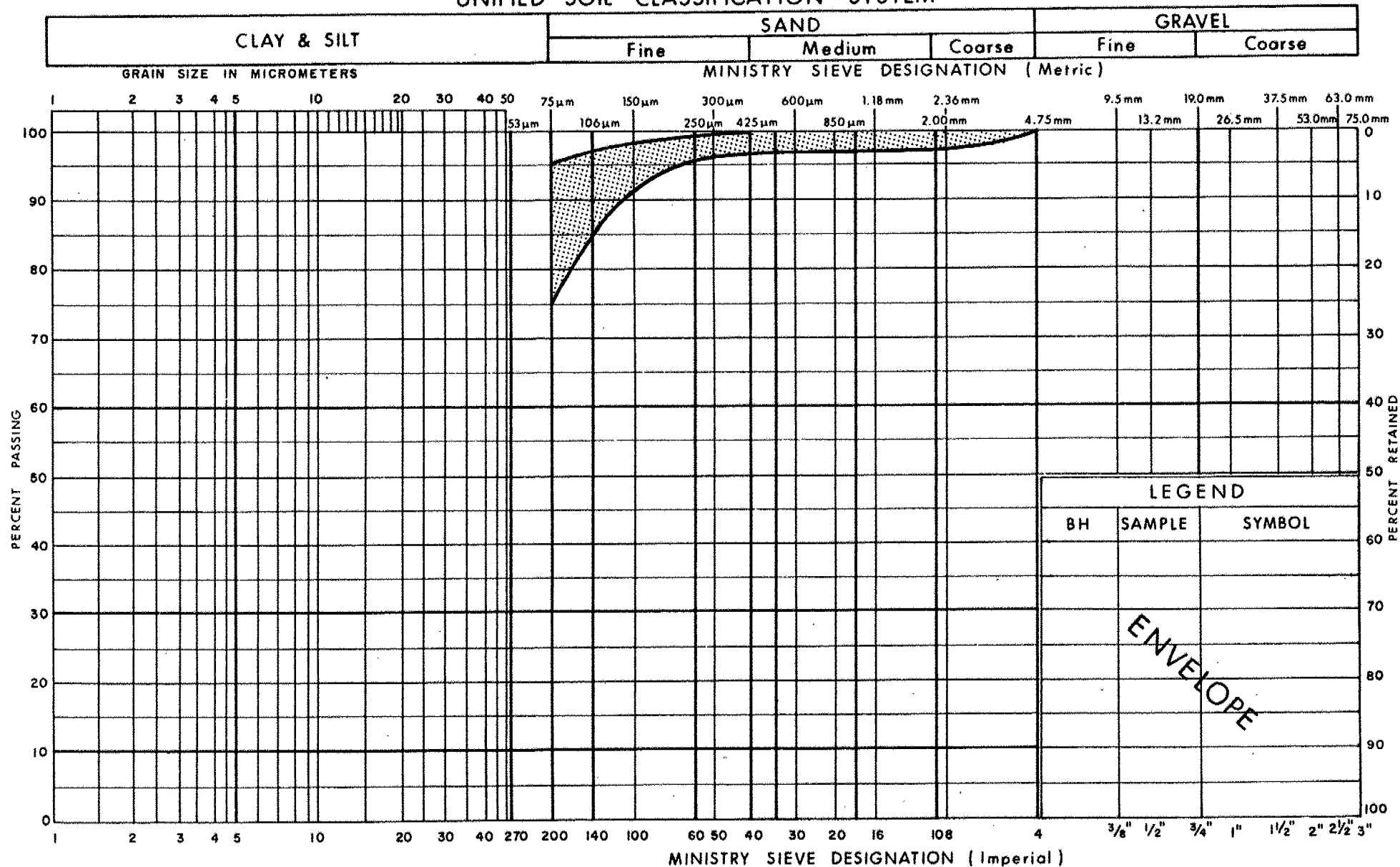
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION SILTY SAND

FIG No 1

W P 26-96-01

UNIFIED SOIL CLASSIFICATION SYSTEM

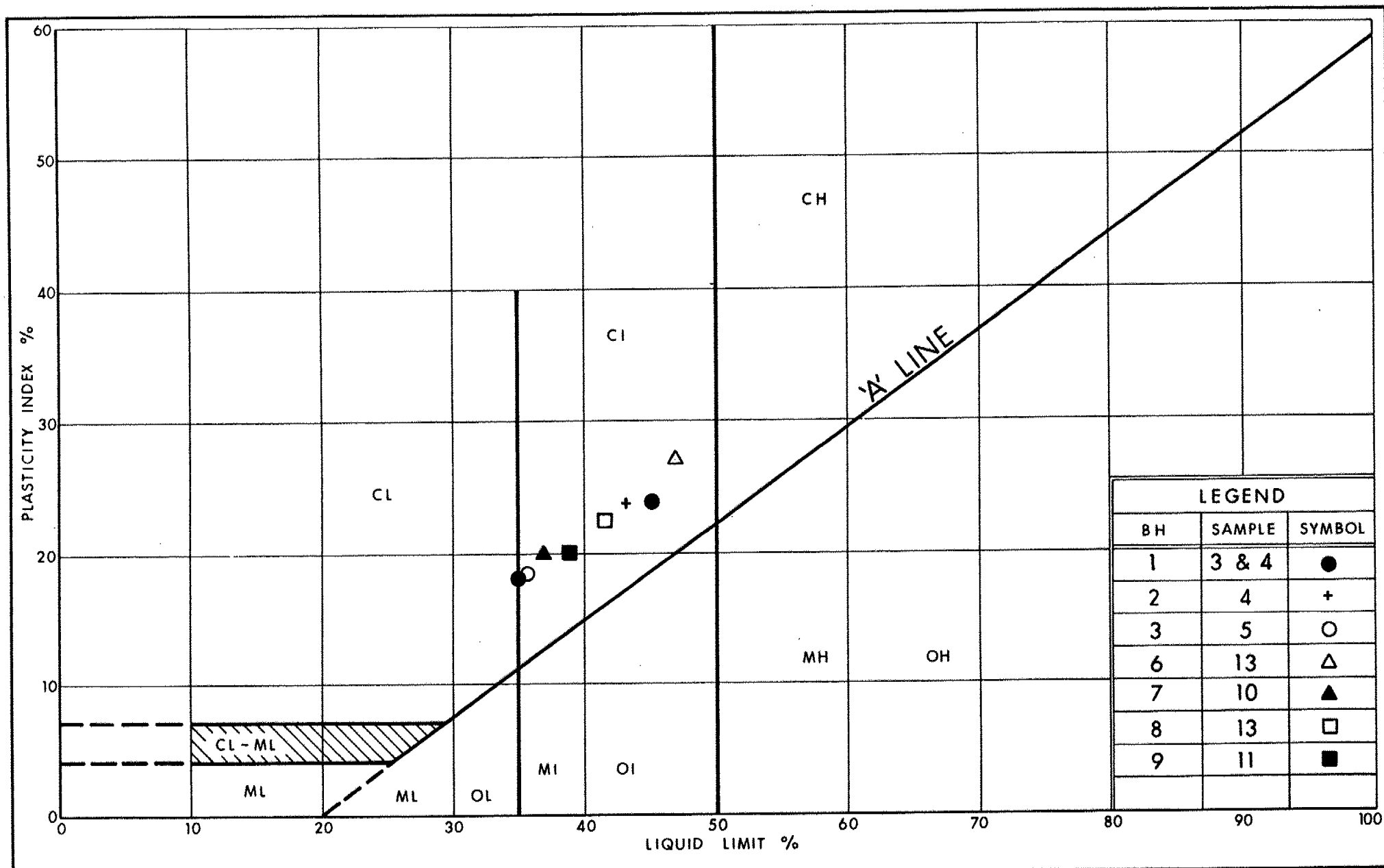

 Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

SILT, TRACE / SOME SAND

FIG No 2

W P 26-96-01



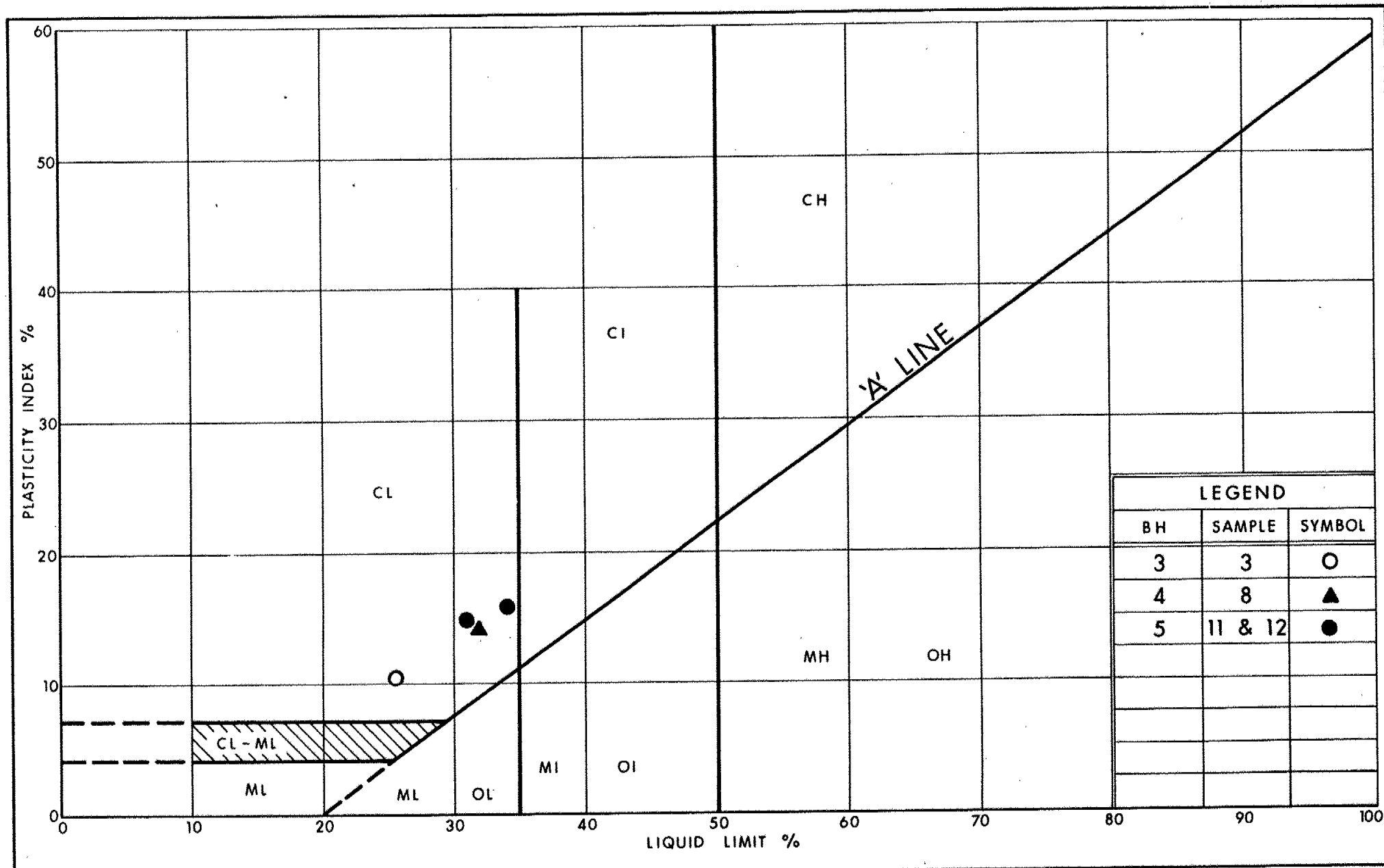
Ministry of
Transportation

Ontario

PLASTICITY CHART SILTY CLAY

FIG No 3

W P 26-96-01



Ministry of
Transportation

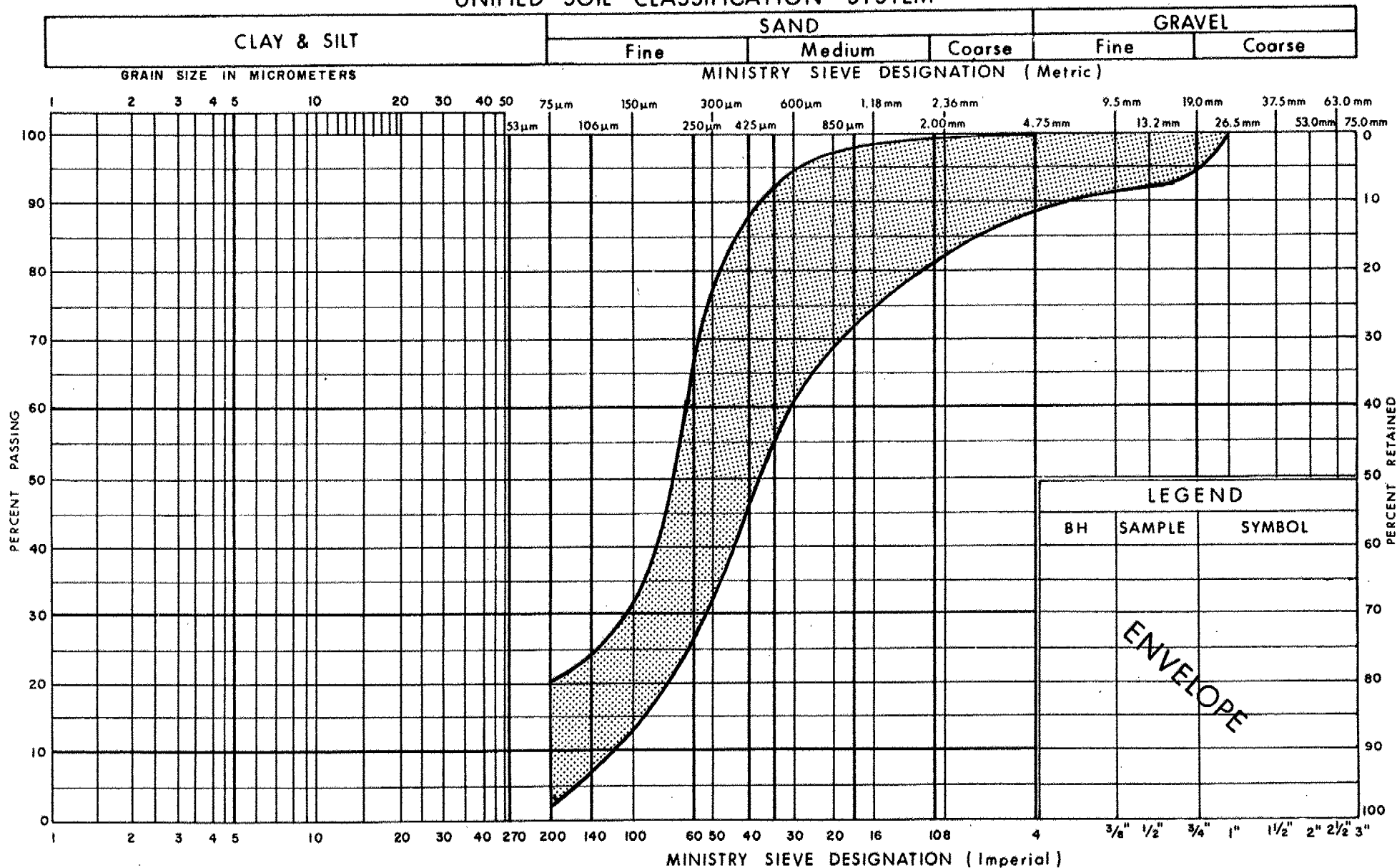
Ontario

PLASTICITY CHART CLAYEY SILT

FIG No 4

W P 26-96-01

UNIFIED SOIL CLASSIFICATION SYSTEM



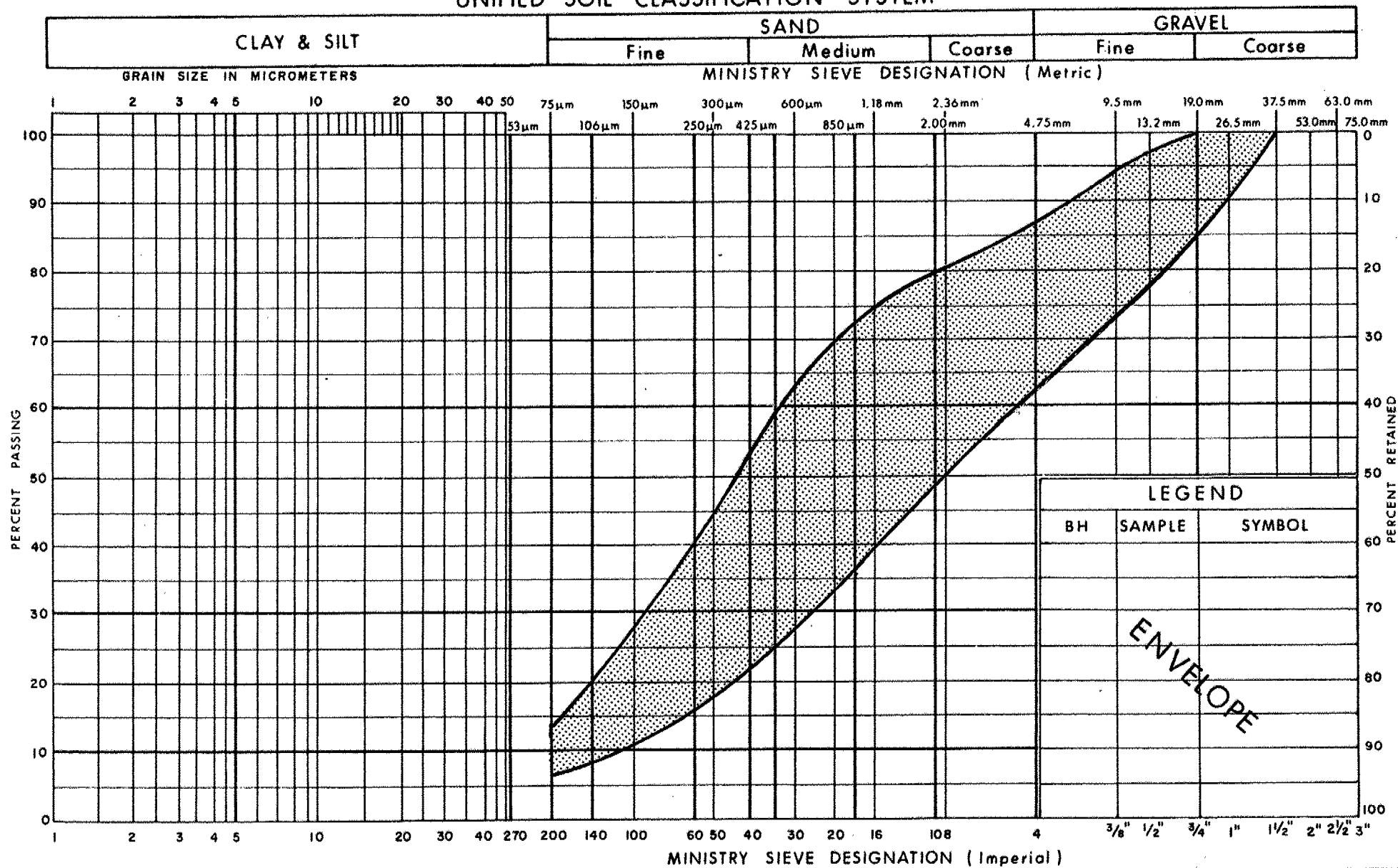
Ministry of
Transportation

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND,
TRACE OF GRAVEL

FIG No 5

W P 26-96-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION GRAVELLY SAND

FIG No 6

W P 26-96-01

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