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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Cubert Street Overpass Structure Site No. 22-174 Highway 401 Improvements from Brock Street to Courtice Road Regional Municipality of Durham W.O. 10-20011

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REPORT



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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
CUBERT STREET OVERPASS
STRUCTURE SITE NO. 22-174
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO
COURTICE ROAD, REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario.

This report addresses the proposed replacement and widening of the Cubert Street overpass. This report was developed with information from a previous geotechnical/foundations investigation at the Cubert Street overpass site, as follows:

- **MTO GEOCREs No. 30M15-029:** "Foundation Investigation Report for the Proposed Widening of Existing Overpass Structure at the Crossing of Highway 401 and Cubert Street, City of Oshawa, Site #22-174, W.O. 71-11010, W.P. 44-71-07", prepared by MTO Foundations Office, dated July 30, 1973.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2010-E-0062 dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of the *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

2.0 SITE DESCRIPTION

The Cubert Street overpass is located at the intersection of Highway 401 and Cubert Street, approximately 6 km east of Brock Road in the City of Oshawa, in the Regional Municipality of Durham (see key plan on Drawing 1). The existing overpass is a single-span structure supported on shallow foundations that was constructed in 1941, and widened in 1975.

The overall surface topography in the vicinity of the overpass site generally slopes gently downward to the east. The local topography is also influenced by a drumlinoid feature that rises some 10 m above the prevailing grade north of Highway 401. The natural ground surface in the immediate vicinity of the Cubert Street overpass is between approximately Elevation 108 m and 114 m, and is typically near Elevation 110 m to 111 m.

Cubert Street was constructed by cutting the local road into the natural ground, approximately 3 m to 8 m below the original ground surface. Beneath Highway 401, Cubert Street is about 6 m below the highway at approximately Elevation 105 m to 106 m. In the vicinity of the overpass, Highway 401 is raised 1 m to 2 m above the original grade to approximately Elevation 111 m to 112 m.

3.0 INVESTIGATION PROCEDURES

Four boreholes were advanced at this site as part of a previous subsurface investigation by MTO in April 1973, in support of the widening of Highway 401 and the Cubert Street overpass at that time (MTO GEOCREs Report No. 30M15-029). For the purposes of the current Preliminary Foundation Investigation Report, the boreholes have been re-numbered such that the 30M15-series GEOCREs number precedes the original borehole number. For example, Borehole 1 from GEOCREs Report No. 30M15-029 is referred to throughout this report and on the drawings as Borehole 029-1. The approximate borehole locations are shown on Drawing 1; these borehole



locations have been interpreted based on scaling measurements from the plan shown in the 1973 GEOCRETS report.

The boreholes were advanced using a CME auger drill. The boreholes extended to depths ranging from 10.5 m to 13.6 m. Standard Penetration Testing (SPT) was carried out at 0.75 m to 3 m depth intervals. Dynamic Cone Penetration Testing (DCPT) was also conducted using comparable practices. Consistent with standard of practice at that time, it is assumed that SPTs and DCPTs were advanced using a manually-operated drop hammer. The water level in the open boreholes was observed during the drilling operations.

Index and classification testing (water content, Atterberg limits and grain size distributions) was completed on selected samples.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984) and *Urban Geology of Canadian Cities* (Brennan, 1998). The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the 1973 investigation, together with the results of in situ and laboratory testing, are presented on the borehole records and laboratory test results provided in Appendix A. Interpreted stratigraphic profiles along the westbound and eastbound lanes are shown on Drawing 1.

The stratigraphic boundaries shown on the borehole records and on Drawing 1 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profile at the structure site, shown on Drawing 1, is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

Summary descriptions of the subsurface conditions encountered in the boreholes are provided in the following sections. In general, the subsurface conditions at the site consist of a surficial cohesive fill layer, underlain by a glacial till deposit of slight to intermediate plasticity.

4.2.1 Fill

Approximately 1.1 m to 2.0 m of fill was encountered immediately below the ground surface at the borehole locations, at the time of the 1973 investigation. The fill was described as clayey silt containing some sand and some gravel.



Standard Penetration Test (SPT) 'N'-values ranging from 7 to 9 blows per 0.3 m of penetration were recorded, with one test recording 1 blow per 0.3 m of penetration. This indicates that the fill has a very soft to stiff consistency. Dynamic Cone Penetration Test (DCPT) records indicate the average blows to penetrate 0.3 m was about 9, with a range of values between 1 and 25.

No laboratory index testing was completed as part of the 1973 investigation for the fill material.

4.2.2 Clayey Silt to Silty Clay Till

A deposit of clayey silt to silty clay till was encountered underlying the fill material in all four boreholes, at Elevations 103.2 m to 104.0 m. All of the boreholes terminated within this deposit, after penetrating it for a depth of approximately 7.5 m to 12.0 m (to approximately Elevation 91.5 m to 94.7 m).

The borehole records describe this till as a heterogeneous mixture of clayey silt, sand and gravel. An envelope of grain size distributions is shown on Figure 1 in Appendix A; based on interpretation of the results shown on the borehole records, it is inferred that not all of the grain size distribution test results are included on this figure. The results of the grain size distributions are summarized in the following table:

Range	Gravel Sizes	Sand Sizes	Silt Sizes	Clay Sizes
Minimum	1	3	24	9
Maximum	14	40	69	73
Typical Range	3-10	28-38	34-42	17-28

Atterberg limits testing was completed on 16 selected samples of the till deposit. On 15 of the samples, the measured plastic limits range from about 10 to 20 per cent, the liquid limits from about 17 to 30 per cent, and the plasticity indices from about 4 to 10 per cent; these results are plotted on a plasticity chart on Figure 2 in Appendix A, and confirm that the tested samples consist of clayey silt of low plasticity. An Atterberg limits test on one sample of this deposit from Borehole 029-1 measured a plastic limit of about 25 per cent, a liquid limit of approximately 50 per cent, and a plasticity index of about 25 per cent; this suggests that this portion of the till deposit would be classified as silty clay of intermediate plasticity. The natural water contents measured on samples of the till range from about 5 to 12 per cent, except for the more plastic silty clay in Borehole 029-1 where a water content of about 25 per cent was measured; these values are typically at or below the plastic limit for the till material.

The measured SPT 'N' values in the clayey silt to silty clay till range from 37 to greater than 100 blows per 0.3 m of penetration, suggesting that the deposit has a hard consistency.



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4.3 Groundwater Conditions

The groundwater level was measured in the open boreholes during the 1973 drilling operations; these measurements are shown on the borehole records in Appendix A, and summarized in the table below:

Borehole No.	Ground Surface Elevation (m)	Depth to water Level in Open Borehole (m)	Groundwater Elevation in Open Borehole (m)	Comments
029-1	105.1	1.6	103.5	During drilling – April 1973
029-2	105.2	2.0	103.2	During drilling – April 1973
029-3	105.1	1.6	103.5	During drilling – April 1973
029-4	105.1	1.1	104.0	During drilling – April 1973

The 1973 groundwater level is near the interface between the fill and the glacial till, below the Cubert Street cut grade which is at about Elevation 105 m. However, the water levels observed in the open boreholes during the investigation may not represent the long-term stabilized groundwater levels, nor the current groundwater regime at the site. The groundwater level should be expected to fluctuate seasonally and should be expected to rise during wet periods of the year.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Andrew Van Dyk, P.Eng., a senior geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
CUBERT STREET OVERPASS
STRUCTURE SITE NO. 22-174
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO
COURTICE ROAD, REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



6.0 DISCUSSION AND PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement and widening of the existing Cubert Street overpass and associated wingwalls/retaining walls. These preliminary recommendations are based on interpretation of the factual data obtained from boreholes advanced at this site in 1973 by MTO. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Design Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

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 or operational constraints may be required in the contract documents in the future. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, the existing single-span Cubert Street overpass will be replaced with a new single-span overpass structure that is both longer and wider.

The existing structure foundation details were obtained from three drawings:

- *Overpass at Cubert Street, General Plan and Elevations*, Contract No. 39.01, Drawing No. D2743, Prepared by MTO, dated April 2, 1941
- *Cubert Street Overpass, General Plan*, WP No. 44-71-07, Site No. 22-174, Sheet 1, prepared by MTO, dated April 1974.
- *Cubert Street Overpass, Foundation Layout and Ret. Walls*, WP No. 44-71-07, Site No. 22-174, Sheet 3, prepared by MTO, dated April 1974.

These drawings indicate the original structure constructed in 1941 was an approximately 13.8 m long rigid-frame, single-span overpass supported on 1.1 m wide spread footings founded at approximately Elevation 103.4 m to 103.5 m. The structure was widened in 1975 by about 2.5 m in both the eastbound and westbound directions, with the widening designed to match the existing structure.

Based on the preliminary General Arrangement (GA) drawing developed by AECOM, the existing structure will be fully removed and replaced with a new structure with an approximately 17 m span. In order to accommodate the widening of Highway 401, the new overpass will be wider by about 12 m on the north side, and 10 m on the south side. Wingwalls will be required adjacent to the abutments, parallel to Highway 401. The Cubert Street grade will be lowered under the new structure, to approximately Elevation 104.3 m.



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Based on the subsurface conditions, both shallow and deep foundation options have been considered for the overpass replacement. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in the table following the text of this report.

- **Strip or spread footings founded on the hard clayey silt till deposit:** Shallow foundations (strip or spread footings) are feasible for support of the new structure provided that they are founded below the fill materials, on the hard clayey silt till. This would require excavation of approximately 1.1 m to 2.0 m relative to the existing Cubert Street grade. Shallow foundations would allow for construction of semi-integral abutments, but would not permit integral abutments. Temporary protection systems will be required along Highway 401, as well as at the Cubert Street grade parallel to the abutment footings, to facilitate the staged removal of the existing structure and construction of the new overpass. The proposed footing founding level will extend near the groundwater level at the site, and some seepage is expected from water perched above the cohesive till near the base of the fill materials.
- **Footings “perched” on a compacted granular pad in the Highway 401 approach embankments:** Footings “perched” in the approach embankments above the Cubert Street grade are feasible for support of the new abutments and associated wing walls. However, a longer structure span would be required to construct abutment foreslopes for an “open” structure configuration, and the structural costs would be much higher. Therefore, this option is not detailed further in this report.
- **Driven steel H-piles or pipe piles founded within the “100-blow” soils:** Driven steel H-piles or steel pipe piles are feasible and suitable for support of the abutments, and would allow for the construction of integral abutments. A perched pile cap in conjunction with integral abutments in a false abutment configuration would minimize and groundwater control requirements at the new abutment locations; the existing Highway 401 embankment would need to be excavated to the new Cubert Street grade (but not to frost depth below this level, as would be the case for spread footings). Hard till soils, with SPT ‘N’-values of greater than 100 blows per 0.3 m of penetration, occur at relatively shallow depth at this site. Pre-augering is recommended to ensure that the piles penetrate to adequate depth, remain aligned, and are not damaged. Relative to H-piles, pipe piles may have a slightly higher risk for “hanging up” or being deflected away from their designed orientation due to the hard nature of the soils and the potential presence of cobbles and/or boulders within the glacially-derived soils at this site. Further, the use of driving shoes is recommended due to the hard nature and the potential presence of cobbles and boulders in the glacially-derived soils.
- **Caissons:** Caissons are feasible for this site. Although they are more expensive than driven pile foundations, the likely requirement for pre-augering at the pile locations will reduce the cost differential. Although not encountered in the existing boreholes at this site, the till deposit may contain lenses and interlayers of water-bearing granular soil; as a result, the use of temporary or permanent liners is recommended to minimize loss of ground and disturbance to the caisson base.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the overpass replacement on shallow foundations, due to the relatively shallow depth to hard glacial till. Driven steel H-pile foundations in an integral abutment configuration are considered a good and feasible alternative from a geotechnical/foundations perspective.



6.3 Shallow Foundations

6.3.1 Founding Elevations

For support of the new abutments, associated wingwalls or retaining walls, strip or spread footings should be founded below the fill and any loose or soft to stiff surficial soils, to bear on the hard clayey silt till deposit. Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). 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.

The Cubert Street grade is presently at about Elevation 105 m below the bridge; its grade is proposed to be lowered to approximately Elevation 104.3 m. The table below provides the maximum (highest) founding elevations recommended for preliminary design of footings founded on the hard clayey silt till deposit; however, the footings will have to be extended deeper, to about Elevation 103.1 m, to achieve the minimum depths for frost protection.

Foundation Element	Strip/Spread Footing Founding Elevation
West abutment	Below 103.5 m (103.1 m for frost protection)
East abutment	Below 103.2 m (103.1 m for frost protection)

The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately, a 100 mm thick concrete working slab (of 20 MPa compressive strength concrete) should be placed on the prepared subgrade within four hours of its inspection and approval.

6.3.2 Geotechnical Resistance/Reaction

For preliminary design of spread/strip footings founded on the properly prepared clayey silt till deposit at or below the elevations provided in Section 6.3.1, the factored geotechnical axial resistances at Ultimate Limit States (ULS) may be taken as 500 kPa. The geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) may be taken as 350 kPa. These values assume a long, concentrically loaded footing wider than 2.5 m. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Table 10.2 in *CFEM* (2006).

The preliminary geotechnical resistance values provided above should be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutments, and the final footing width and founding elevation.

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

6.4.1 Founding Elevations

The abutments and associated wing walls/retaining walls may be supported on steel H-piles or steel pipe piles driven to found within the hard clayey silt till deposit, which was generally found to have SPT 'N' values greater than 100 blows per 0.3 m of penetration. For preliminary design purposes, if integral abutments are adopted for the design of the structure replacement, it has been assumed that the pile caps would be “perched” within the



approach embankments, with the underside of the new pile caps at approximately Elevation 108 m. Given the relatively shallow depth to 100-blow soil below the Cubert Street grade, pre-augering will be required. A pile tip elevation of 99 m may be used for preliminary design purposes, following at least 3 m of pre-augering and assuming approximately 3 m of penetration into the “100-blow” soil deposit. This design tip elevation does not account for pile embedment length required to achieve desired lateral pile performance.

The hard nature of the “100-blow” glacial soils, potentially containing cobbles and boulders, may subject driven piles to a higher risk of “hanging up” or being deflected away from their design orientation during installation. In this regard, steel H-piles are preferred over steel pipe piles. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSP 3000.100 (*Steel H-Pile Driving Shoe*) or OPSP 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard ‘H’ Bearing Pile Points) are preferred over flange plates.

6.4.2 Axial Geotechnical Resistance/Reaction

HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1 can be designed using a factored axial resistance at ULS of 1,400 kN, and an axial geotechnical reaction at SLS (for 25 mm of settlement) of 1,200 kN. Similar axial resistances may be used in the design for 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.)

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). Termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile type and length; the criteria must therefore be established at the time of construction after the piling equipment is known. Pile capacity can be verified in the field using the Hiley formula (MTO Standard Structural Drawing SS-103-11) during the final stages of driving to achieve an appropriate ultimate capacity.

The preliminary 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 new abutment foundation elements.

6.5 Retained Soil System (RSS) Walls

If perched pile caps are used in a false abutment configuration, and for retaining walls adjacent to the abutments and wingwalls at this site, retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations. In fact, they are advantageous in that they would minimize the depth of excavation below the Cubert Street grade, as compared to strip footings.

6.5.1 Founding Elevations

The facing panels and the reinforced soil mass of the RSS walls should be founded below any existing topsoil or unsuitable fill soils. Typically, the front facing panels are supported on a footing and/or granular levelling pad at a shallow depth below the ground surface in front of the wall. It is recommended that the facing panels be founded at a minimum depth of 0.5 m below the lowest surrounding grade, in accordance with MTO’s *RSS Design Guidelines*. The levelling pad should consist of a minimum thickness of 0.3 m of compacted OPSS.PROV 1010 Granular A, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.



6.5.2 Geotechnical Resistance/Reaction

For the RSS facing panels founded on compacted granular fill as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

Assuming that the RSS wall (up to approximately 8 m high) acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.7 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 500 kPa and a geotechnical reaction at SLS of 350 kPa may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised if necessary during the design-build assignment after the RSS wall configuration and any “step” elevations are confirmed, taking into account any additional subsurface information at that time.

6.5.3 Global Stability of RSS Walls

A global stability analysis has been performed, using the program *Slide 6.0* by RocScience Inc., to check that a minimum factor of safety of 1.5 is achieved for a proposed retaining wall under static conditions. This minimum factor of safety is considered appropriate for this project, considering the design requirements and the available field and laboratory testing data.

The parameters listed in the table below have been used in the analyses, based on field and laboratory test data from the 1973 investigation, as well as generally accepted correlations. The parameters for the fill and native soils above the Cubert Street cut grade (i.e., above the top elevation of the 1973 boreholes) have been assumed and will require confirmation through further investigation during detailed design.

Soil Stratum	Bulk Unit Weight (kN/m ³)	Effective Friction Angle
Fill and/or native soils above Cubert Street cut grade (behind RSS wall)	21	30°
Hard (100-blow) clayey silt till	22.5	35°

The subsoils at this site are generally comprised of hard, “100-blow” clayey silt till. The factor of safety against global instability of RSS walls is greater than 1.5. This preliminary assessment of the global stability of the retaining walls should be reviewed and confirmed as part of the detailed design, once the wall geometry (in particular the presence and height of any sloping ground) is refined and further borehole information is obtained within the footprint of the walls.

The internal stability of the reinforced earth structure is to be assessed by the proprietary product supplier.

6.5.4 Settlement

The Highway 401 embankments are relatively low (on the order of 1 m to 2 m relative to the natural ground surface above the Cubert Street cut grade). For the proposed highway embankment widening, and assuming a nominal grade raise of up to approximately 0.5 m, it is estimated that the settlement in the underlying soils will be less than 25 mm. This settlement is expected to be completed during construction. Based on this preliminary estimate, we anticipate the settlement performance for RSS walls and facing panels will be acceptable.



However, this preliminary assessment of embankment/RSS wall settlement should be reviewed and confirmed based on the subsoil conditions encountered beneath the proposed retaining walls (and particularly boreholes advanced from above the Cubert Street cut grade) during detail design.

6.6 Approach Embankments

The existing Highway 401 approach embankments are approximately 1 m to 2 m high relative to the natural ground surface at the site. The Cubert Street cut will be deepened from approximately Elevation 105 m to 104.3 m under the highway, and the cut will be widened in conjunction with the overpass replacement.

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that any topsoil and surficial soft/loose soils should be stripped from the footprint of the Highway 401 approach embankment widening. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available via boreholes advanced from the natural ground surface above the Cubert Street cut grade.

Benching the existing embankment side slopes should be carried out in accordance with OPSD 208.010 (*Benching of Earth Slopes*). The widened Highway 401 embankment side slopes should be formed at a maximum gradient of 2H:1V. In accordance with MTO's standard practice, a bench at least 2 m wide should intervene embankment side slopes greater than 8 m in total height, and cut slopes greater than 6 m in height; although the embankment and cut configuration at this site may fall below these thresholds, consideration should be given to incorporation of a horizontal area in the transition between the highway embankment side slope and the Cubert Street cut slope to improve the surficial stability/erosion performance of the overall slope system. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod should be carried out as soon as practicable after construction of the embankments.

6.6.2 Approach Embankment Stability

As the existing Highway 401 embankment is only 1 m to 2 m high relative to the natural ground surface at the site, detailed global stability analyses have not been completed. However, these low embankments are expected to have factors of safety for global stability of greater than 1.3, subject to confirmation of the nature of the soils within the approach embankment footprint during detailed design.

6.6.3 Approach Embankment Settlement

Settlement analyses for the anticipated soil conditions below the widened approach embankments were carried out using the estimated elastic deformation moduli in the table below. These are based on generally accepted correlations with the SPT 'N' values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Conditions	Bulk Unit Weight (kN/m³)	Elastic Modulus (MPa)
Native soils above Cubert Street grade (assumed to consist of very stiff soils)	21	20-30
Very stiff to hard clayey silt till	22.5	150



Based on this preliminary assessment, the settlement of the foundation soils under the widened 1 m to 2 m high approach embankments is estimated to be less than 25 mm is expected to occur relatively quickly during and immediately following construction of the widened approach embankments. This estimated magnitude of settlement should be reassessed based on the soil conditions under the widened approach embankments as determined during the detail design, with particular emphasis on the thickness and properties of any surficial soil deposits within the embankment widening footprint.

The foregoing does not include compression of the fill itself, which would occur during and after embankment construction. The magnitude of fill compression will depend on the type of fill used and may range from 0.5 to 1.0 per cent of the embankment height; granular fill will tend compress essentially during embankment construction, whereas fine-grained earth fill will tend to exhibit some additional settlement over time. This assumes the fill is properly moisture conditioned and compacted to about 98 per cent of the material's standard Proctor maximum dry density.

6.7 Construction Considerations

The following sections identify construction considerations that may impact the future detail design, and for which provision may be required in the contract documents produced as part of detail design.

6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings would extend to a depth of about 2 m below the Cubert Street cut grade, to extend below the base of fill materials and provide adequate frost protection. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The soils within the excavation depth below Cubert Street grade would be classified as Type 3 soil, according to the OHSA. Additional investigation will be required in detail design to classify the soils below the Highway 401 grade; however, at this preliminary stage, it is anticipated that these soils will be above the groundwater level and would also be classified as Type 3 soils. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

Temporary protection systems will be required along the existing Highway 401 eastbound and westbound lanes to facilitate the removal of the existing bridge foundations and staged construction of the new, longer-span structure. Shallow temporary protection systems may also be required parallel to Cubert Street, to facilitate removal of the existing abutments and construction of the new abutments. Temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Construction Specification for Temporary Protection Systems*). Lateral movements of the temporary shoring should meet Performance Level 2 as specified in OPSS.PROV 539, provided that the existing adjacent Highway 401 structures, as well as any adjacent utilities, can tolerate this magnitude of deformation.

Driven, interlocking sheetpiles or soldier piles with timber lagging are common methods for temporary excavation support in the Greater Toronto Area. An interlocking sheetpile system is likely not feasible for this site as it would be difficult to install into the "100-blow" clayey silt till that was encountered in the 1973 boreholes immediately below the Cubert Street grade. A pre-augered soldier pile and lagging system is likely more feasible from a constructability perspective, but it may require measures to mitigate loss of soil particles through the lagging boards if seepage from perched groundwater is encountered above the Cubert Street grade.



6.7.2 Groundwater Control

The 1973 investigation noted water in the open boreholes was near the interface between the base of the Cubert Street fill and the surface of the clayey silt till. However, the water levels observed in the open boreholes during the investigation may not represent the long-term stabilized groundwater levels, nor the current groundwater regime at the site. Further assessment of the local groundwater conditions should be conducted as part of future investigations in detailed design; however, based on the relatively low permeability of the clayey silt till deposit, the seepage volume is expected to be relatively small, such that the water inflow can be handled by pumping from filtered sumps placed at the base of the excavations. A Permit to Take Water (PTTW) should not be required for the groundwater control system in this case.

6.7.3 Subgrade Protection

The clayey silt till soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the foundation 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.

6.7.4 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or shoring systems. The frequency of occurrence of cobbles and boulders should be identified during future investigations as part of the detail design, to permit the contractor to assess the impact on foundation construction. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the potential presence of cobbles and/or boulders within the overburden soils.

6.7.5 Vibration Monitoring During Pile Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition, with lower threshold values applicable for other structures or buildings (typically between 25 mm/s and 50 mm/s). If pile driving is adopted at the abutments, then vibration monitoring is recommended adjacent to the abutment areas to demonstrate/confirm that vibration levels do not exceed the thresholds.

6.8 Recommendations for Further Work in Detail Design

Additional boreholes will be required within each of the foundation widening areas and the approach embankment widening areas during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Abutments and retaining walls:
 - Assessment of the upper clayey silt till properties to confirm the founding elevation and geotechnical resistances/settlement performance for spread footings and RSS walls.
 - Assessment of the depth and properties of the “100 blow” clayey silt till to confirm pile tip elevations, if deep foundations are selected as the preferred option.



PRELIMINARY FOUNDATION REPORT CUBERT STREET OVERPASS, W.O. 10-20011

- Observation of the presence and frequency of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile foundations and protection systems.
- Widened highway embankments:
 - Assessment of the depth and extent of stripping of topsoil/organics and fill materials within the footprint of the widened embankments.
 - Further assessment of the thickness and consolidation/elastic compression properties of the soils within the footprint of the widened embankments (above the Cubert Street grade), to confirm the settlement estimates.
- Walls and temporary protection systems:
 - Assessment of the properties of the existing embankment fill and native soils above the Cubert Street grade to estimate lateral earth pressures for abutment and wall design, and to provide information for the contractor to design temporary protection systems.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Andrew Van Dyk, P.Eng., a senior geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.



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Senior Geotechnical Engineer



Lisa Coyne, P.Eng.
Principal, Designated MTO Foundations Contact

AVD/LCC/sm

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PRELIMINARY FOUNDATION REPORT CUBERT STREET OVERPASS, W.O. 10-20011

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario

Other

Ontario Regulation 213	Construction Projects (as amended)
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PRELIMINARY FOUNDATION REPORT CUBERT STREET OVERPASS, W.O. 10-20011

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES – CUBERT STREET OVERPASS

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings on hard clayey silt till	<ul style="list-style-type: none"> Feasible for support of new abutments and concrete retaining walls 	<ul style="list-style-type: none"> Existing structure supported on shallow foundations, and has performed well Relatively minor groundwater seepage anticipated Very dense soils (with SPT 'N' values of greater than 100 blows per 0.3 m of penetration) present at shallow depth, with good geotechnical resistances and settlement performance Allows for semi-integral abutments Lower vibration impacts on existing structures than for driven steel H-pile or steel pipe (tube) pile installation 	<ul style="list-style-type: none"> Precludes use of integral abutments; potentially greater maintenance required at abutments More significant protection systems required through Highway 401 than for a deep foundation option with perched pile cap 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration Estimated cost is about \$600/m³ for a concrete unit for construction of shallow foundations, excluding deeper excavation and temporary protection system
Spread/strip footings perched on compacted granular pad in approach embankment fill	<ul style="list-style-type: none"> Feasible but not recommended for support of a new overpass, as a longer span structure would be required in an open configuration 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings founded below Cubert Street grade, reducing excavation depth and associated temporary protection system requirements 	<ul style="list-style-type: none"> Not compatible with a "closed" structure configuration; longer span with abutment foreslopes or RSS would be required 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 	<ul style="list-style-type: none"> This option not recommended due to longer span required



PRELIMINARY FOUNDATION REPORT CUBERT STREET OVERPASS, W.O. 10-20011

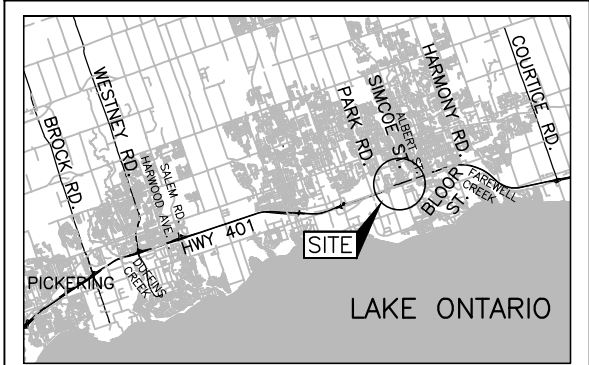
Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Driven steel H-piles or pipe piles	<ul style="list-style-type: none"> • Feasible for support of new abutments • Not required for support of retaining walls 	<ul style="list-style-type: none"> • Allows for integral abutment construction if existing structure can be modified to accommodate, or if replacement is adopted • Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support compared to shallow foundation option • Limited groundwater control required 	<ul style="list-style-type: none"> • Potential for deviated piles in hard "100 blow" soils; pre-augering will likely be required • Potential for encountering obstructions (cobbles and/or boulders) during pile driving; this could result in piles "hanging up" and lower geotechnical resistances • Potential for noise and/or vibration impacts on nearby commercial buildings • Increase costs for pre-augering to reduce potential for deviated and/or damaged piles. 	<ul style="list-style-type: none"> • Conventional construction methods for H-pile foundations • Pre-augering or churning may be required. 	<ul style="list-style-type: none"> • Lower relative cost compared with caisson option • Estimated unit cost is approximately \$250/linear metre for pile installation and \$600/m³ for pile cap construction, plus additional costs associated with pre-augering
Caissons founded in hard clayey silt till	<ul style="list-style-type: none"> • Feasible for support of abutments, but more expensive than shallow foundations or driven piles • Not required for support of retaining walls 	<ul style="list-style-type: none"> • Abutment pile caps could be maintained higher than footings founded on till deposit, reducing depth of excavation and temporary excavation support requirements compared to shallow foundation option • Higher capacity than for steel H-piles, so reduced number of deep foundation elements compared to steel H-piles 	<ul style="list-style-type: none"> • Temporary or permanent liners would be required due to potential for water-bearing granular layers within the clayey silt till; likely not possible to inspect caisson base • Precludes use of integral abutments 	<ul style="list-style-type: none"> • Minor to moderate risk of loosening water-bearing granular layers if encountered 	<ul style="list-style-type: none"> • Higher cost compared with shallow foundations or steel H-piles

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

CONT No.
WO No. 10-20011

CUBERT STREET OVERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS

SHEET



LEGEND

1973 Boreholes - GEOCRES No. 30M15-029

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
029-1	105.1	4860193.4	355935.7
029-2	105.2	4860197.4	355947.9
029-3	105.1	4860163.4	355960.0
029-4	105.1	4860159.0	355946.5

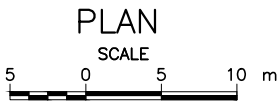
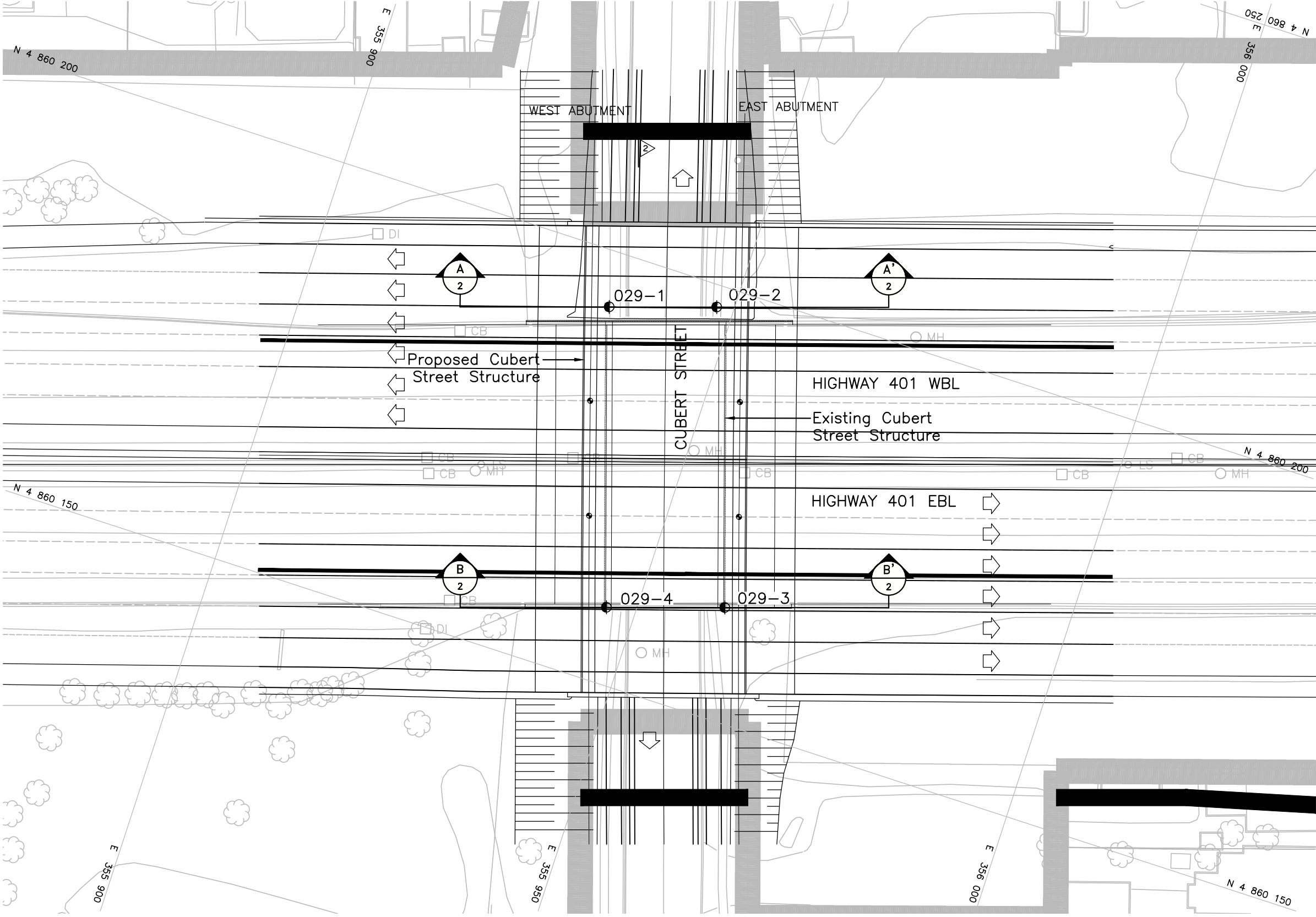
NOTES

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 Preliminary Design Report.

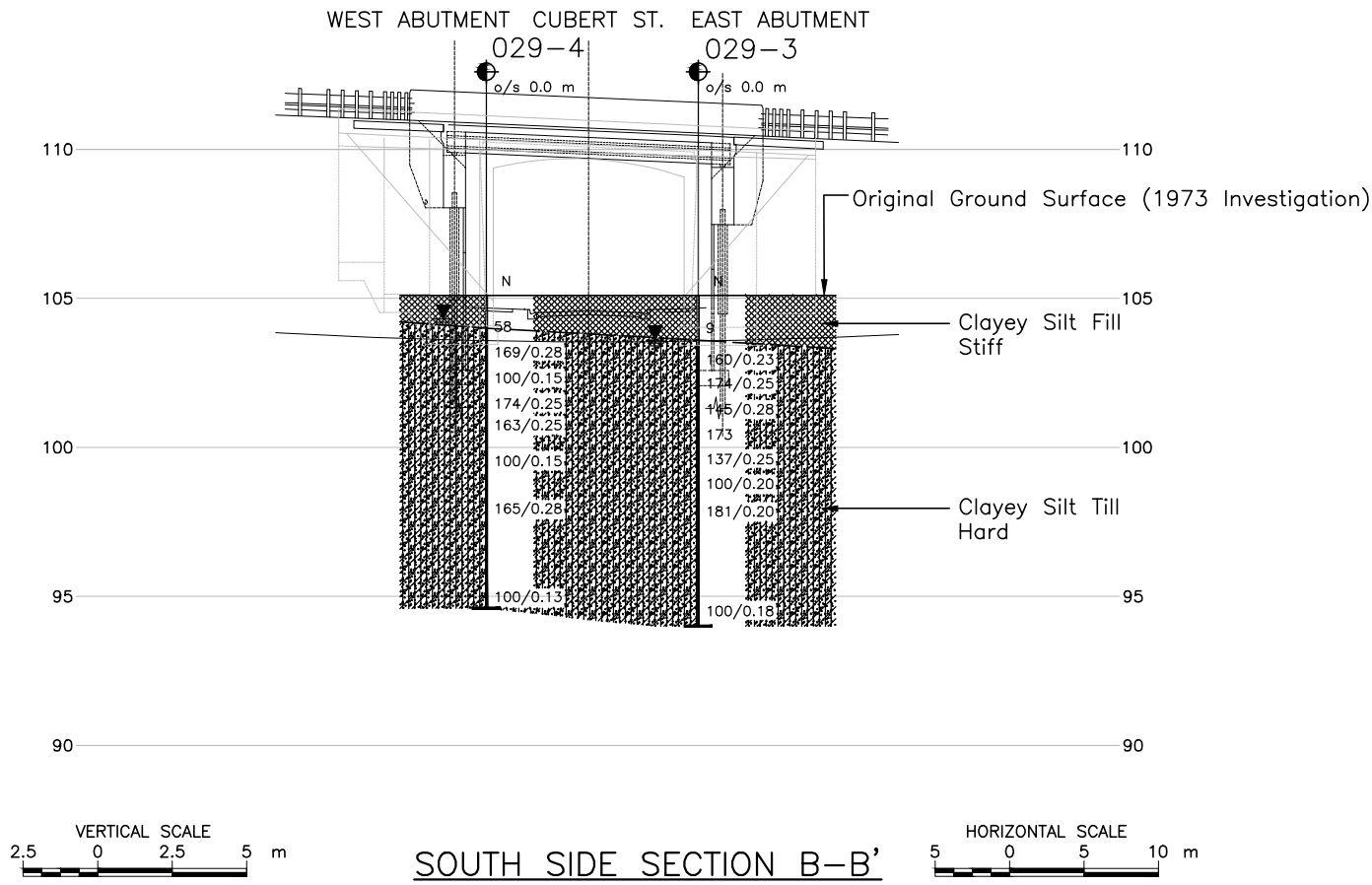
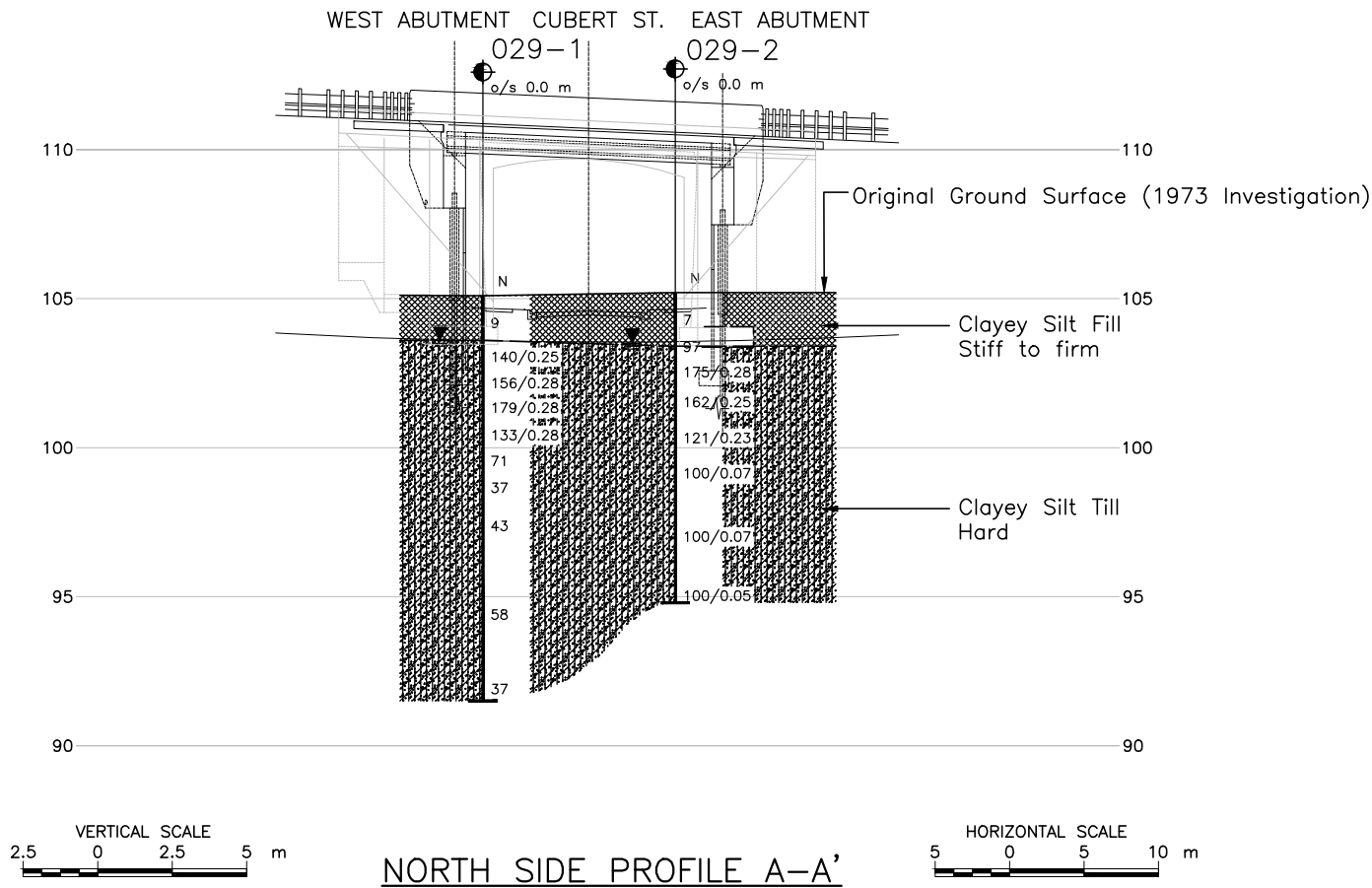
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg, X-Property.dwg and Street Names.dwg, and the Proposed Design obtained from drawing file x-design_130625.dwg, all dated July 05, 2013, received April 11, 2014. General Arrangement provided in digital format by AECOM file no. ACAD-CubertSt_GA Overpass.dwg, received October 25, 2016.



NO.	DATE	BY	REVISION	
Geocres No. 30M15-299				
HWY. 401		PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. AVD		CHKD. LCC	DATE: 4/5/2017	SITE: 22-174
DRAWN: JFC/DD		CHKD. AVD	APPD. LCC	DWG. 1

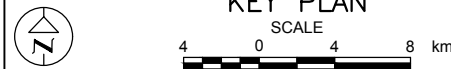
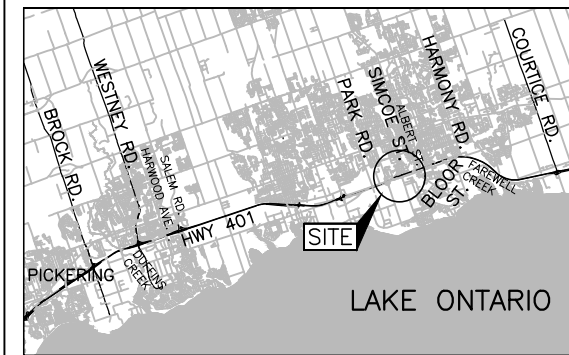


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 10-20011

CUBERT STREET OVERPASS
HIGHWAY 401 IMPROVEMENTS
SOIL STRATA

SHEET



LEGEND

1973 Boreholes - GEOCREs No. 30M15-029

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
029-1	105.1	4860193.4	355935.7
029-2	105.2	4860197.4	355947.9
029-3	105.1	4860163.4	355960.0
029-4	105.1	4860159.0	355946.5

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 Preliminary Design Report.

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

REFERENCE

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NO.	DATE	BY	REVISION
Geocres No. 30M15-299			
HWY. 401	PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. AVD	CHKD. LCC	DATE: 4/5/2017	SITE: 22-174
DRAWN: JFC/DD	CHKD. AVD	APPD. LCC	DWG. 2



APPENDIX A

Borehole Records and Laboratory Test Results 1974 Investigation (GEOCRES No. 30M15-029)

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 73-11010

LOCATION Co-ords. 15,944,784 N & 1,167,715 E

ORIGINATED BY V.K.

W.P. 44-71-07

BORING DATE April 17, 1973.

COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L	
344.7	Ground level.														
0.0	Clayey silt with some sand and gravel.														
339.7	(Fill material) Stiff		1	SS	9	340									(103.5m)
5.0			2	SS	140	10"									WL 339.7
	Heterogeneous mixture of clayey silt to silty clay, sand and gravel.		3	SS	156	11"									9-37-35-19
			4	SS	179	11"									4-37-40-19
			5	SS	133	11"									
	(Glacial till)		6	SS	71										0-3-24-73
			7	SS	37										
	Hard		8	SS	43	320									
			9	SS	58	310									
300.2			10	SS	37	300									2-28-42-28
44.5	End of Borehole														

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

JOB 73-11010

LOCATION Co-ords. 15,944,798N & 1,167,761E

ORIGINATED BY V.K.

W.P. 44-71-07

BORING DATE April 18, 1973

COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
345.0	Ground level															
0.0	Clayey silt with some sand and gravel and traces of organics		1	SS	7	340										(103.2m)
338.5	(Fill material) Firm		2	SS	97											WL 3385 1-21-69-9
6.5	Heterogeneous mixture of clayey silt, sand and gravel		3	SS	175	11"										3-39-41-17
	(Glacial fill)		4	SS	162	10"										
			5	SS	121	9"	330									
			6	SS	100	3"										1-32-48-19
	Hard		7	SS	100	3"	320									
310.8			8	SS	100	2"	310									
34.2	End of Borehole															

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 73-11010

LOCATION Co-ords. 15,944,684N & 1,167,798E

ORIGINATED BY V.K.

W.P. 44-71-07

BORING DATE April 19, 1973

COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Auger and Core Test

CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L		
344.7	Ground level															
0.0	Clayey silt with some sand and gravel and traces of organics (fill material) Stiff		1	SS	9	340										(103.5m) WL 339.7
339.7			2	SS	160/9"											14-34-34-18
5.0	Heterogeneous mixture of clayey silt, sand and gravel		3	SS	74/0"											
			4	SS	145/1"	330										
			5	SS	173											
	(Glacial till)		6	SS	377/0"											
			7	SS	100/4"											
			8	SS	81/6"	320										4-38-40-18
	Hard															
308.2			9	SS	100/7"	310										
36.5	End of Borehole															
						300										

(m)
105.1

103.5
1.6

93.9
11.2

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE NO 4

FOUNDATIONS OFFICE

JOB 73-11010

LOCATION Co-ords. 15,944,669N & 1,167,752E

ORIGINATED BY V.K.

W.P. 44-71-07

BORING DATE April 24, 1973

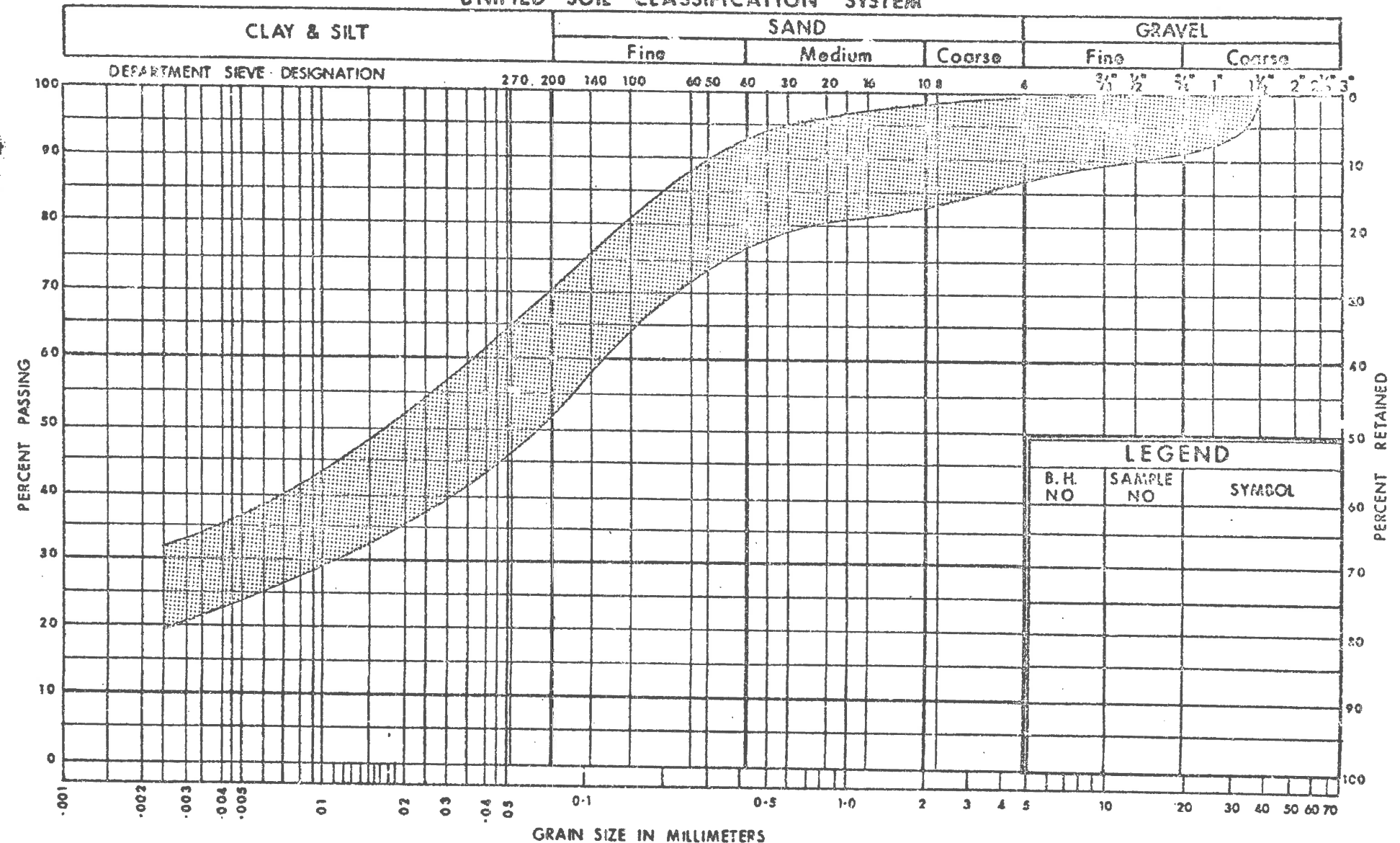
COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	w_p	w	w_L	
105.1 (m) (ft)	344.7 Ground level														
104.0 1.1	0.0 Clayey silt with some sand and gravel 341.2 (Fill material) Stiff		1	SS	58	340									(104.0m) 341.2
	3.5 Heterogeneous mixture of clayey silt, sand and gravel (Glacial till)		2	SS	169	11"									6-38-38-18
			3	SS	100	6"									
			4	SS	174	10"									12-33-35-20
			5	SS	163	10"									
			6	SS	100	6"									
	Hard		7	SS	165	11"									4-40-37-10
94.5 10.6	310.2 End of Borehole		8	SS	100	5"									

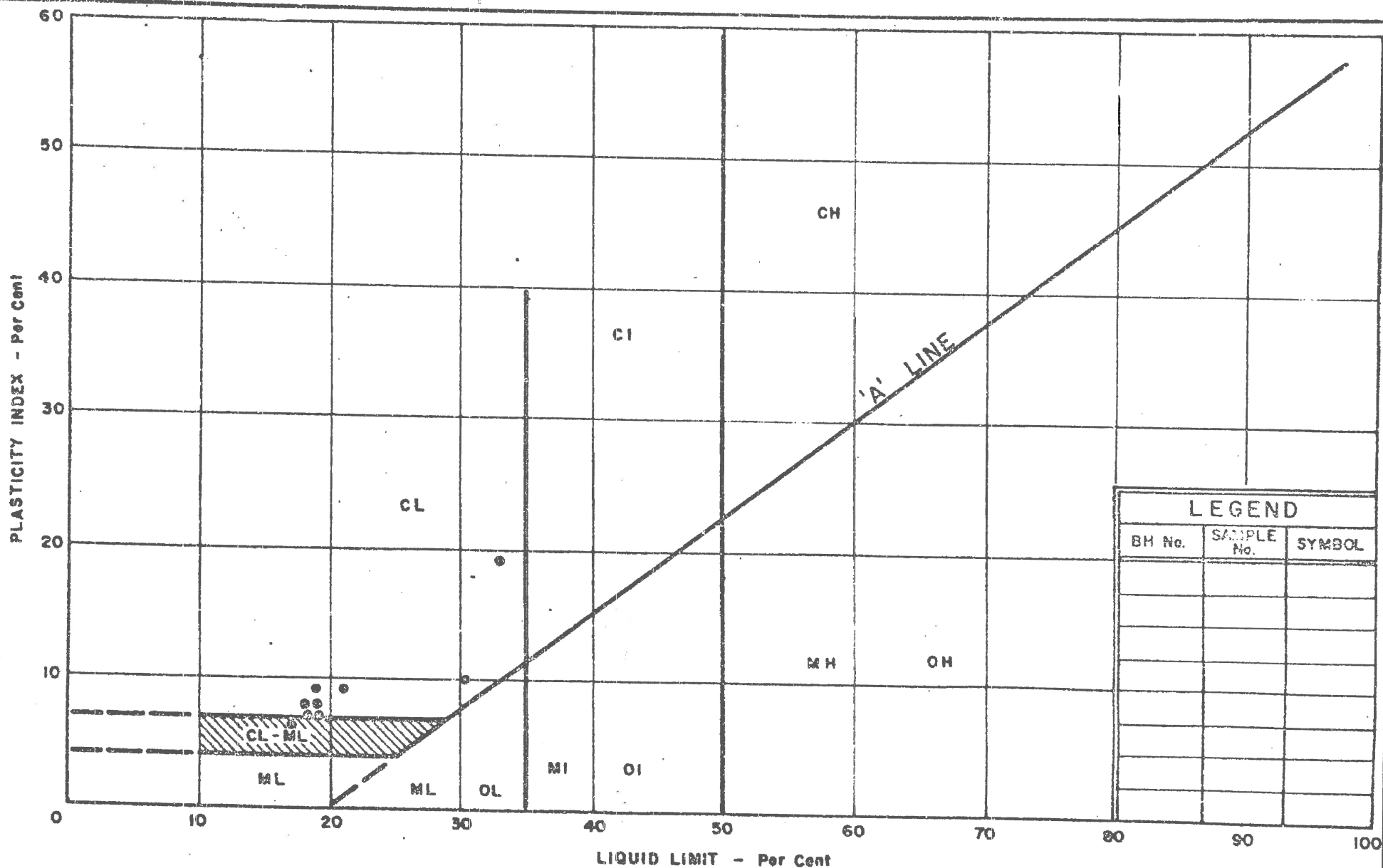


GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.P. No. 44 - 71 - 07

JCS No. 73 - 11010

FIG. 1



LEGEND		
BH No.	SAMPLE No.	SYMBOL



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART
GLACIAL TILL
HET. MIXTURE OF CLAYEY SILT, SAND & GRAVEL

WP. No. 44-71-07

JOB No. 73-11010

FIG. 2

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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