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## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

### Ritson Road Overpass Structure Site No. 22-179 Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

**Submitted to:**

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REPORT



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# **PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
RITSON ROAD OVERPASS  
STRUCTURE SITE NO. 22-179  
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 improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario. This report addresses the proposed replacement of the existing Ritson Road overpass.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposals (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 AECOM's *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 Ritson Road overpass is located between Albert Street and Wilson Road South in the City of Oshawa, in the Regional Municipality of Durham. The existing Ritson Road overpass is a single-span structure supported on spread footings, with a span of about 12.8 m. A pedestrian tunnel extends under the Highway 401 embankment immediately east of the east abutment, and a retaining wall is present along the east side of the east sidewalk.

The natural ground surface in the vicinity of this structure site varies from about Elevation 100 m to 101 m. Ritson Road has been constructed in a shallow cut, about 2 m to 3 m deep, with its grade at approximately Elevation 98 m beneath the overpass. The Highway 401 grade over the structure site is at approximately Elevation 103.5 m, with approach embankments that are approximately 3 m in height. A commercial property is located in the northwest quadrant of the structure site, and single-family residences are present both north and south of the highway.

## **3.0 INVESTIGATION PROCEDURES**

Four boreholes were advanced at this site as part of a previous geotechnical/foundation investigation by MTO in 1973, in support of the widening of the Ritson Road overpass (MTO GEOCRETS Report No. 30M15-31). For the purposes of the current Preliminary Foundation Investigation Report, the boreholes have been re-numbered such that the 30M15-series GEOCRETS number precedes the original borehole number. For example, Borehole 1 from GEOCRETS Report No. 30M15-31 is referred to throughout this report and on the drawings as Borehole 31-1. 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 1973 boreholes were drilled by a track-mounted drill rig using continuous flight augers, from the Ritson Road cut grade. Soil sampling was performed at 0.75 m and 1.5 m intervals of depth using a 50 mm outside diameter split-spoon sampler driven by a manual hammer, in accordance with the Standard Penetration Test (SPT) procedure. The groundwater conditions were observed in the open boreholes during drilling.

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* (Brennand, 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 figures 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.

In general, thin layers of topsoil and fill (present at the time of the 1973 investigation) are underlain by a very dense sandy silt to silty sand till deposit. This upper till deposit is underlain by a hard silty clay deposit, which is in turn underlain by very dense sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### **4.2.1 Fill**

Fill material was encountered in Borehole 31-4 at the time of the 1973 investigation, immediately below a thin layer of topsoil. The fill was encountered at a depth of about 0.3 m, and was approximately 1.1 m thick, with its base at about Elevation 97.0 m. The fill consists of clayey silty containing sand and gravel.

One Standard Penetration Test (SPT) "N"-value of 4 blows per 0.3 m of penetration was measured within the fill, suggesting that this material has a soft to firm consistency.

#### **4.2.2 Silty Sand to Sandy Silt Till**

A non-cohesive till deposit was encountered below a thin layer of topsoil and fill (where present). The till deposit extends to depths of 8.2 m to 9.8 m below the ground surface at the time of the investigation, with the deposit base between approximately Elevation 90.3 m and 88.6 m.

The till deposit consists of sandy silt to silty sand, containing trace clay and trace gravel. An envelope of grain size distribution test results completed on eight samples of the till deposit are presented on Figure 1 in Appendix A. Atterberg limits testing was completed on six samples of the till, and measured plastic limits of 10 to 11 per



cent, liquid limits of 13 to 15 per cent, and plasticity indices of 3 to 4 per cent; one sample was determined to be non-plastic. The measured natural water contents range from approximately 5 to 8 per cent, generally below the plastic limit for the deposit.

The measured SPT “N”-values within the silty sand to sandy silt till range from 57 blows per 0.3 m to 100 blows per 0.075 m of penetration, indicating a very dense relative density.

#### **4.2.3 Silty Clay**

A silty clay deposit was encountered underlying the till, at a depth of about 8.2 m to 9.7 m (Elevation 90.3 m to 88.6 m) in Boreholes 31-1 to 31-4. The deposit was fully penetrated in Boreholes 31-2 and 31-4, where it is 1.9 m and 3.7 m in thickness, with its base at about Elevation 87.9 m and 84.9 m, respectively. Boreholes 31-1 and 31-3 were terminated within the silty clay deposit, after penetrating it for 2.9 m to 4.1 m.

Atterberg limits testing was completed on seven samples of this deposit, and measured plastic limits of 18 to 24 per cent, liquid limits of 35 to 42 per cent, and plasticity indices of 15 to 18 per cent. These results, which are plotted on a plasticity chart on Figure 2 in Appendix A, confirm that this deposit consists of silty clay of intermediate plasticity. The measured natural water contents range from about 15 to 24 per cent, typically slightly below the plastic limit for the material.

The measured SPT “N”-values within the silty clay range from 56 blows to 159 blows per 0.3 m of penetration, indicating a hard consistency.

#### **4.2.4 Lower Sand**

A lower sand deposit was encountered below the silty clay deposit, at a depth of about 10.6 m and 13.5 m (Elevation 87.9 m and 84.9 m) in Boreholes 31-2 and 31-4, respectively. These boreholes were terminated within the sand deposit after penetrating it for a thickness of about 2.2 m to 5.1 m.

The deposit consists of sand containing trace silt and trace gravel. The results of grain size distribution tests on two selected samples are shown on Figure 3 in Appendix A. The natural water contents measured on two samples of the sand were 18 and 19 per cent.

The measured SPT “N”-values within the sand ranged from 62 blows to 130 blows per 0.3 m of penetration, indicating a very dense relative density.

### **4.3 Groundwater Conditions**

The groundwater levels were measured in the open boreholes during the 1973 drilling operations. Details of the measured groundwater levels are shown on the borehole records in Appendix A, and summarized below:

<b>Borehole No.</b>	<b>1973 Ground Surface Elevation (m)</b>	<b>Depth to Water Level (m)</b>	<b>Groundwater Elevation (m)</b>	<b>Date</b>
31-1	98.5	0.5	98.0	July 12, 1973
31-2	98.5	0.6	97.9	July 13, 1973
31-3	98.5	0.5	98.0	July 13, 1973
31-4	98.4	0.7	97.7	July 16, 1973



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These 1973 groundwater levels may not represent the stabilized groundwater level at the site, nor the current groundwater regime. The groundwater level will be subject to seasonal fluctuations and precipitation events. The water level should be expected to be higher during the spring season, and following periods of heavy precipitation or snow melt.

### 5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Pat Speirs and was reviewed by Ms. Nikol Kochmanová, P.Eng., a 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  
RITSON ROAD OVERPASS  
STRUCTURE SITE NO. 22-179  
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD  
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## **6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides preliminary foundation recommendations in support of the proposed replacement of the existing Ritson Road overpass (MTO Structure Site 22-179). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 1973 subsurface investigation at this site. 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 the construction or design-build contractor. 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. 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 Ritson Road overpass will be replaced with new single-span overpass structures that is both longer and wider.

The existing overpass was constructed in 1941, and widened by about 6 m on both the south and north sides in 1975. Based on the available design drawings, the existing structure is supported on spread footings that are founded at approximately Elevation 96.7 m (317.4 ft.), about 1.2 m to 1.3 m below the current Ritson Road grade.

Based on the preliminary General Arrangement (GA) drawing provided by AECOM, Ritson Road is proposed to be widened from two to four lanes plus pedestrian sidewalks on both sides, resulting in a new longer span length of about 28 m. Highway 401 and the new overpass will be widened by about 5 m on the north side and about 8 m on the south side. New wingwalls/retaining walls will be required parallel to Highway 401, to minimize impacts on the adjacent commercial and residential properties. The Highway 401 grade is proposed to be increased by about 0.2 m to 0.3 m at the structure site, with the new grade at approximately Elevation 103.7 m. The Ritson Road grade will be lowered by approximately 1.5 m, to approximately Elevation 96.5 m.

Both shallow and deep foundation options have been considered for support of the new Ritson Road overpass. 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 Table 1 following the text of this report.

- **Strip or spread footings founded on the very dense sandy silt to silty sand till deposit:** Shallow footings are feasible at this site due to the very dense relative density of the near-surface soils. This foundation type would preclude the use of integral abutments, but could permit semi-integral abutments. This option would require excavation to a depth of about 1.2 m below the new, lowered Ritson Road grade. Temporary



protection systems will be required along Highway 401, as well as at the Ritson Road grade parallel to the abutment/retaining wall footings, to facilitate the removal of the existing structure and the construction of the new overpass. The proposed footing founding level will extend below the groundwater level at the site (as measured during the 1973 investigation), and groundwater control is expected to be required in the sandy silt to silty sand till to stabilize the subgrade and enable shallow foundations to be constructed in “dry” conditions.

- **Footings “perched” on a compacted granular pad in the approach embankment:** At the abutments, footings “perched” in the approach embankments above the Ritson Road 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 (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for pipe piles) or integral abutments (for H-piles). A perched pile cap in conjunction with integral abutments in a false abutment configuration would minimize excavation and groundwater control requirements at the new abutment locations; the existing Highway 401 embankment would need to be excavated to just below the proposed lowered Ritson Road grade, but not as deep as would be required for spread footings. Very dense “100-blow” soil will be encountered at shallow depths below the Ritson Road grade, and pre-augering may be required to ensure that the piles penetrate to adequate depth, remain aligned, and are not damaged. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense till deposit.
- **Caissons founded within the “100-blow” soils:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. This option will be more expensive than either shallow foundations or pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required. If caissons are adopted for support of the abutments, they would extend into and through water-bearing non-cohesive till, and temporary liners would be required during construction to control potential ground losses and/or disturbance at the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments, although pile foundations are preferred from a geotechnical/foundations perspective as they would permit integral abutments, and a perched pile cap would reduce excavation and groundwater control requirements as compared with spread footings for support of a closed structure configuration.

## **6.3 Shallow Foundations**

### **6.3.1 Founding Elevation and Frost Protection Requirements**

For support of the abutments and associated wingwalls and concrete retaining walls (if adopted) for the new overpasses, spread/strip footings should be founded on the very dense sandy silt to silty sand till deposit. Concrete retaining walls beyond the wingwalls may also be founded on the native soils above the Ritson Road cut grade, although further borehole investigation will be required during detailed design to assess the nature and properties of these soils and confirm the applicable foundation recommendations. 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



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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.

Ritson Road is presently at about Elevation 98 m, and the grade is proposed to be lowered to approximately Elevation 96.5 m. As such, the maximum (highest) founding elevation recommended for the preliminary design of the abutment and wall footings is approximately Elevation 95.3 m.

### 6.3.2 Geotechnical Axial Resistance and Reaction

The following factored geotechnical axial resistances at Ultimate Limit States (ULS) and geotechnical reactions at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of spread/strip footing founded on the properly prepared sandy silt to silty sand till deposit. These recommendations apply for the new abutment footings, and for retaining wall footings founded below the Ritson Road grade, and are based on an assumed 3 m footing width. Lower geotechnical resistances may apply for the retaining walls above the Ritson Road grade, although further investigation of the soils above this level will be required in detailed design.

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
Footing on properly prepared very dense sandy silt to silty sand till	500	450

Note: The geotechnical resistance/reaction values given above are estimated for a 3 m wide strip footing.

The geotechnical resistances provided above are given for loads will that 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 Table 10.2 in *CFEM* (2006). These preliminary geotechnical resistances will have to be re-evaluated during detail design, subject to additional borehole and groundwater information within the footprint of shallow foundation elements, if adopted.

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

### 6.4.1 Founding Elevation

The abutments and associated wingwalls for the replacement structure may be supported on steel piles driven to found within the “100-blow” sandy silt to silty sand till deposit. Based on the 1973 borehole results, the pile tip elevation is expected to vary. Although Boreholes 31-2 and 31-3 on the north side of Highway 401 encountered 100-blow soil throughout nearly their full depth, Borehole 31-4 in the southwest corner of the structure site encountered 100-blow soils deeper, and Borehole 31-1 in the southeast quadrant encountered a 3 m thick zone of 100-blow soil relatively high, before the SPT “N”-values decreased; refer to the interpreted stratigraphic cross-sections on Drawing 1. Further investigation will be required at the proposed abutment locations during detailed design, to confirm the tip elevation for pile foundations.

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 Ritson Road approach embankments, at about Elevation 100 m. The existing Highway 401 approach embankments and adjacent ground would have to be excavated to near the new, lowered Ritson Road grade (approximately Elevation 96.5 m) for construction of the



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RSS walls including placement of corrugated steel pipe (CSP) sections. Given the shallow depth to 100-blow soil, pre-augering is expected to be necessary prior to driving the piles. The following pile tip elevations are recommended for preliminary design.

Foundation Element	Approximate Elevation Range of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)
WBL Structure – West and East Abutments	97 to 88	91
EBL Structure – West and East Abutments	95 to 93 and Below 89 m in Borehole 031-4	87

Based on the above elevations, the proposed piles are estimated to be approximately 9 m to 13 m long, although longer pile lengths may be determined based on additional investigation during detailed design.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. 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 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to the design tip elevations given above, the factored axial geotechnical resistance at ULS may be taken as 1,600 kN. The axial geotechnical reaction at SLS may be taken as 1,400 kN for 25 mm of settlement. The same axial resistances may be used in the design of 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.). These preliminary geotechnical resistances/reactions will have to be re-evaluated and modified, as necessary, during the detailed design in consideration of the pile cap elevation and additional subsurface investigation at the foundation elements.

Pile installation should be in accordance with OPSS.PROV 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*) during the final stages of driving to verify that the required ultimate capacity has been achieved.



## **6.5 Caisson Foundations**

### **6.5.1 Founding Elevation**

Caissons founded within the “100-blow” till soils may be considered for support of the abutments for the proposed replacement structure. The following caisson founding elevations may be used for preliminary design purposes:

<b>Foundation Element</b>	<b>Approximate Elevation Range of “100-Blow” Soil (m)</b>	<b>Estimated Design Base Elevation (m)</b>
WBL Structure – West and East Abutments	97 to 88	91
EBL Structure – West and East Abutments	95 to 93 and Below 89 m in Borehole 031-4	87

The caissons will extend into and through water-bearing sandy silt to silty sand till, and potentially into water-bearing lower sand deposit in the southern portion of the structure (based on the results from Boreholes 031-1 and 031-4). The groundwater pressure associated with the lower sand deposit is not known based on the results from the 1973 investigation, and further assessment of this aspect will be required during detailed design if caissons are adopted, to confirm requirements for basal stability. If caisson foundations are adopted, a temporary liner will be required to support the overburden soils during construction, in conjunction with drilling mud to balance groundwater pressures to control base disturbance. In addition, placement of concrete by tremie methods would be required.

### **6.5.2 Geotechnical Axial Resistance/Reaction**

The following factored geotechnical axial resistance at ULS and the geotechnical reaction at SLS (for 25 mm of settlement) may be used for design of caisson foundations:

<b>Caisson Diameter (m)</b>	<b>Factored Geotechnical Axial Resistance at ULS (kN)</b>	<b>Geotechnical Reaction at SLS for 25 mm of Settlement (kN)</b>
0.9	2,500	2000
1.2	4,500	3,500

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

## **6.6 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 through the existing Highway 401 embankment and surrounding soils to just below the Ritson Road grade, as compared to the greater depth required for frost protection for strip footings.

### **6.6.1 Founding Elevations**

The front facing panels and the reinforced soil mass of the RSS wall 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.

### **6.6.2 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.

The maximum RSS wall height in front of or adjacent to the abutments and wing walls is estimated to be on the order of 6 m. Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa (for 25 mm of settlement) may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised during detail design after the RSS wall configuration and any "step" elevations are confirmed, taking into account additional subsurface information above the Ritson Road cut grade.

### **6.6.3 Global Stability of RSS Walls**

Preliminary slope stability analyses have been performed for conceptual RSS walls adjacent to the east and west abutments using the commercially available program *Slide 6.0*, produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

<b>Soil Deposit</b>	<b>Bulk Unit Weight (kN/m<sup>3</sup>)</b>	<b>Effective Friction Angle</b>	<b>Undrained Shear Strength (kPa)</b>
Embankment fill and soils above Ritson Road cut grade (assumed)	21	32°	-
Very dense sandy silt to silty sand till	21	35°	-
Hard silty clay	20	32°	-
Very dense sand	21	35°	-





The results of the static global stability analyses indicate that a minimum factor of safety of 1.5 is achieved for RSS walls up to approximately 6 m in retained soil height, assuming level ground in front of and behind the wall, as shown on Figure 1. This preliminary assessment of the global stability of RSS walls should be reviewed and confirmed as part of the detail 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, to characterize the soils above the Ritson Road cut grade.

It should be noted that the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer.

#### **6.6.4 Settlement**

At this preliminary stage, it is estimated that for widened, approximately 3 m high approach embankments on Highway 401, including a nominal grade raise of approximately 0.2 m to 0.3 m, the settlement of the underlying soils will be less than about 10 mm to 15 mm. This settlement is expected to be completed essentially during construction. Based on these estimates, it is anticipated that the settlement performance for RSS walls and facing panels will be acceptable.

### **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 Open-Cut Excavation and Temporary Protection Systems**

The construction of the spread/strip footings would require excavations to about 1.2 m below the new, lowered Ritson Road grade, which is approximately 2.7 m below the current grade. If integral abutment foundations are adopted, excavation will still be required to near the new lowered Ritson Road grade for construction of RSS walls. These excavations will be made through the existing Highway 401 embankment fill and native till deposit, and they will extend below the groundwater table. The existing fill and native soils above the Ritson Road grade are most likely classified as a Type 3 soil, while the native very dense/hard till deposits are classified as Type 2 soils, according to the Occupational Health and Safety Act (OHSA). As such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

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 Ritson Road. 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.

The selection and design of the protection system will be the responsibility of the Contractor.

#### **6.7.2 Groundwater Control**

The groundwater level was measured in the 1973 investigation between Elevation 97.7 m and 98.0 m, near or just below the existing Ritson Road grade. It is anticipated that perched pile caps for an integral abutment can be





maintained above the groundwater level, but that excavations for new retaining wall footings or RSS walls will extend to near or below the groundwater level, given the proposed lowering of Ritson Road by approximately 1.5 m, to about Elevation 96.5 m. However, additional borehole investigation is recommended to confirm the current groundwater level at the site during detailed design.

At this preliminary stage, it is anticipated that an active dewatering system (such as a system of well points or eductors) will be required to lower the groundwater in the fine-grained till deposit to approximately 0.5 m to 1 m below the proposed wall founding level, to maintain a stable subgrade during the construction of footings and/or the reinforced soil mass. An accurate prediction of the groundwater pumping volumes cannot be made based on the 1973 borehole information; in addition, the flow rate would be dependent on whether the contractor includes an interlocking sheetpile cut-off wall and the duration for which the foundation excavation is open. However, it is considered that it may be possible to maintain pumping volumes at less than 50 m<sup>3</sup>/day.

At this preliminary stage, it is anticipated that the zone of influence for the dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the very dense soils that are present at this site. However, the potential for settlement impacts on the existing or new structure foundations and any adjacent utilities should be re-assessed at the detailed design phase.

### **6.7.3 Subgrade Protection**

The native soils that will be exposed within the excavations at the subgrade level for concrete foundations 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 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 protection systems. Further observation is recommended in any future investigation at this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction.

### **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; lower thresholds are applicable for nearby residential and commercial facilities (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 Future Work During Detail Design**

During the detail design phase, additional geotechnical/foundation investigation is recommended to confirm or assess the following:

- The elevation of the “100-blow” soil within the footprint of the proposed abutment locations, to confirm the design pile tip elevations; this is particularly important near the southern portion of the structure, where Borehole 31-1 (in the southeast corner of the structure site) did not encounter 100-blow soil at depth.



## PRELIMINARY FOUNDATION REPORT RITSON ROAD OVERPASS, W.O. 10-20011

- The subsurface conditions above the Ritson Road grade within the footprint of the retaining walls and the embankment widening areas, to confirm the geotechnical resistances, settlement assessment and global stability.
- The current groundwater levels at the site, for more detailed assessment of the groundwater control requirements and measures during construction.

### 7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng.. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

#### GOLDER ASSOCIATES LTD.



Nikol Kochmanová, P.Eng.  
Geotechnical Engineer



Lisa Coyne, P.Eng.  
Designated MTO Foundations Contact

PKS/NK/LCC/sm

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## PRELIMINARY FOUNDATION REPORT RITSON ROAD OVERPASS, W.O. 10-20011

### REFERENCES

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- Brennand, T.A. 1998. Urban Geology Note: Oshawa Ontario. In P.F. Karrow, and O. L. White (Eds.), Geological Association of Canada, Special Paper 42: Urban Geology of Canadian Cities, p. 353-364.
- Canadian Geotechnical Society, 1992. *Canadian Foundation Engineering Manual*, 3<sup>rd</sup> Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
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- Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3<sup>rd</sup> Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.
- Kulhawy, F.H. and Mayne, P.W., 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- NAVFAC, 1982. *Design Manual DM 7.2: Soil Mechanics, Foundation and Earth Structures*. U.S. Navy. Alexandria, Virginia.

### 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 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 RITSON ROAD OVERPASS, W.O. 10-20011

**TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES**

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> <li>Feasible for support of the new abutments, and for support of concrete retaining walls</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques</li> <li>Very dense soils (with SPT “N” values greater than 100 blows per 0.3 m of penetration) present at shallow depth, with good geotechnical resistance and settlement performance</li> </ul>	<ul style="list-style-type: none"> <li>Excavation will extend below the groundwater level and groundwater control will be required</li> <li>Significant temporary protection systems required through Highway 401, with shorter protection systems likely required parallel to Ritson Road</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$600/m<sup>3</sup> for construction of shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Risk of softening/ loosening of footing subgrade; groundwater control is critical</li> </ul>
Driven steel H-piles or pipe piles	<ul style="list-style-type: none"> <li>Feasible for support of abutments</li> <li>Not required for support of retaining walls</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction methods for H-pile or steel pipe pile foundations</li> <li>Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option</li> <li>Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration</li> </ul>	<ul style="list-style-type: none"> <li>Temporary protection systems still required along Highway 401, but may be slightly shallower than those for concrete footings (i.e., to subgrade level for RSS walls rather than footing level)</li> <li>Due to the shallow depth to “100-blow” material, pre-augering will likely be required</li> </ul>	<ul style="list-style-type: none"> <li>Estimated cost is approximately \$250/m length for pile installation and \$600/m<sup>3</sup> for pile cap construction</li> </ul>	<ul style="list-style-type: none"> <li>Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving</li> <li>Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving</li> </ul>




## PRELIMINARY FOUNDATION REPORT RITSON ROAD OVERPASS, W.O. 10-20011

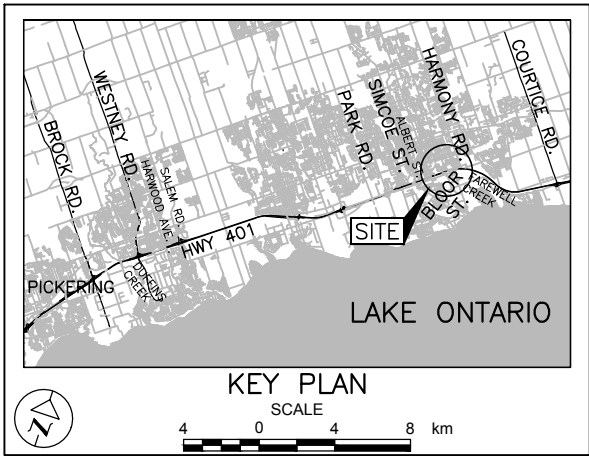
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none"><li>• Feasible but not recommended for support of abutments</li></ul>	<ul style="list-style-type: none"><li>• Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option</li><li>• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles</li></ul>	<ul style="list-style-type: none"><li>• Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance</li><li>• Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base</li><li>• Precludes use of integral abutments</li></ul>	<ul style="list-style-type: none"><li>• Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m<sup>3</sup> for pile cap construction; the cost may be higher to account for temporary liners</li></ul>	<ul style="list-style-type: none"><li>• Risk of loosening or disturbing founding soils at base of caissons</li></ul>

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

CONT No.  
WO No. 10-20011

RITSON ROAD OVERPASS  
HIGHWAY 401 IMPROVEMENTS  
BOREHOLE LOCATIONS

  
SHEET



LEGEND			
	1973 Boreholes - GEOCRETS No. 30M15-031		

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
031-1	98.5	4860607.7	357328.7
031-2	98.5	4860643.2	357316.5
031-3	98.5	4860638.1	357299.3
031-4	98.4	4860601.5	357311.4

**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.

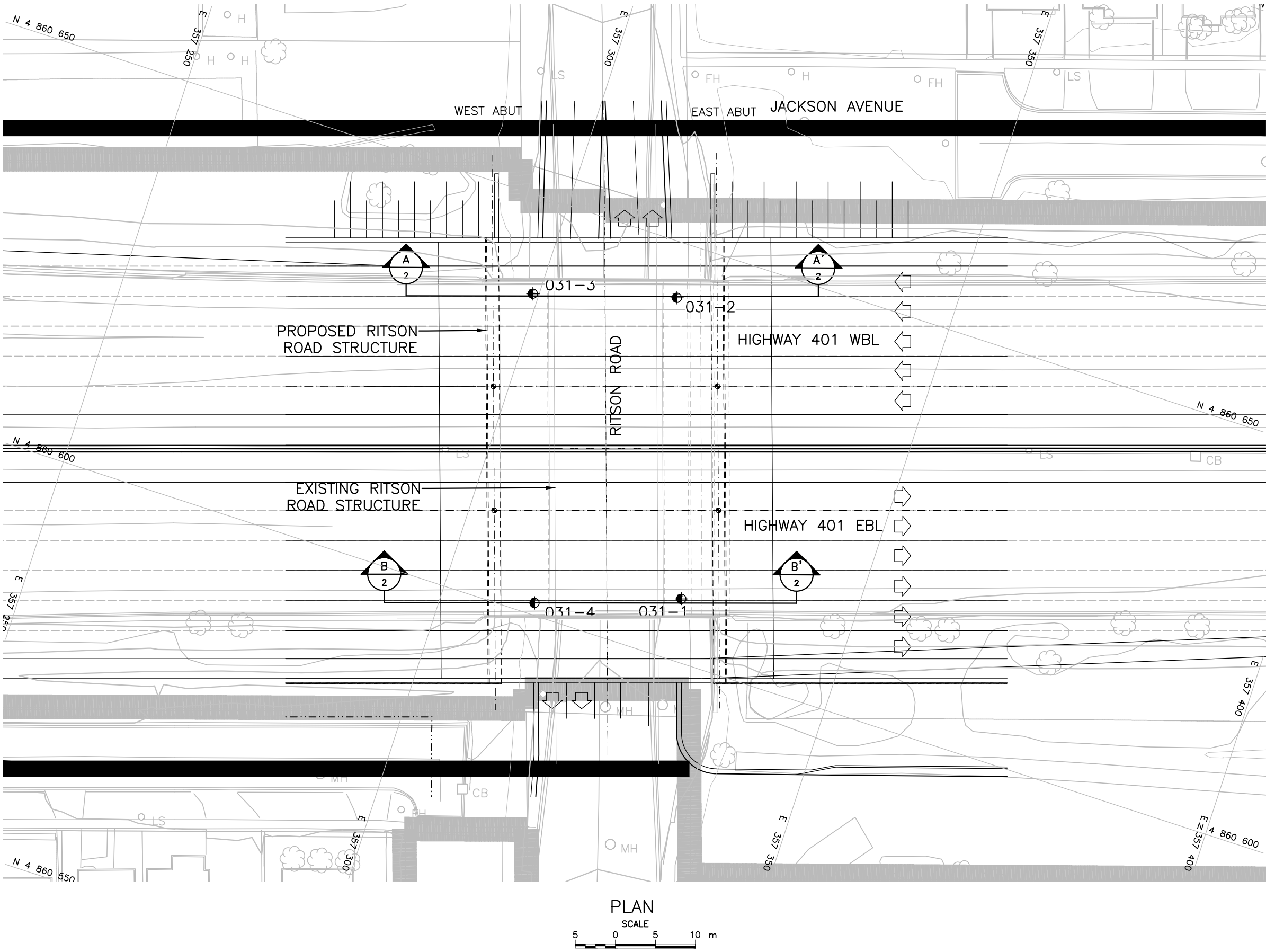
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

**REFERENCE**

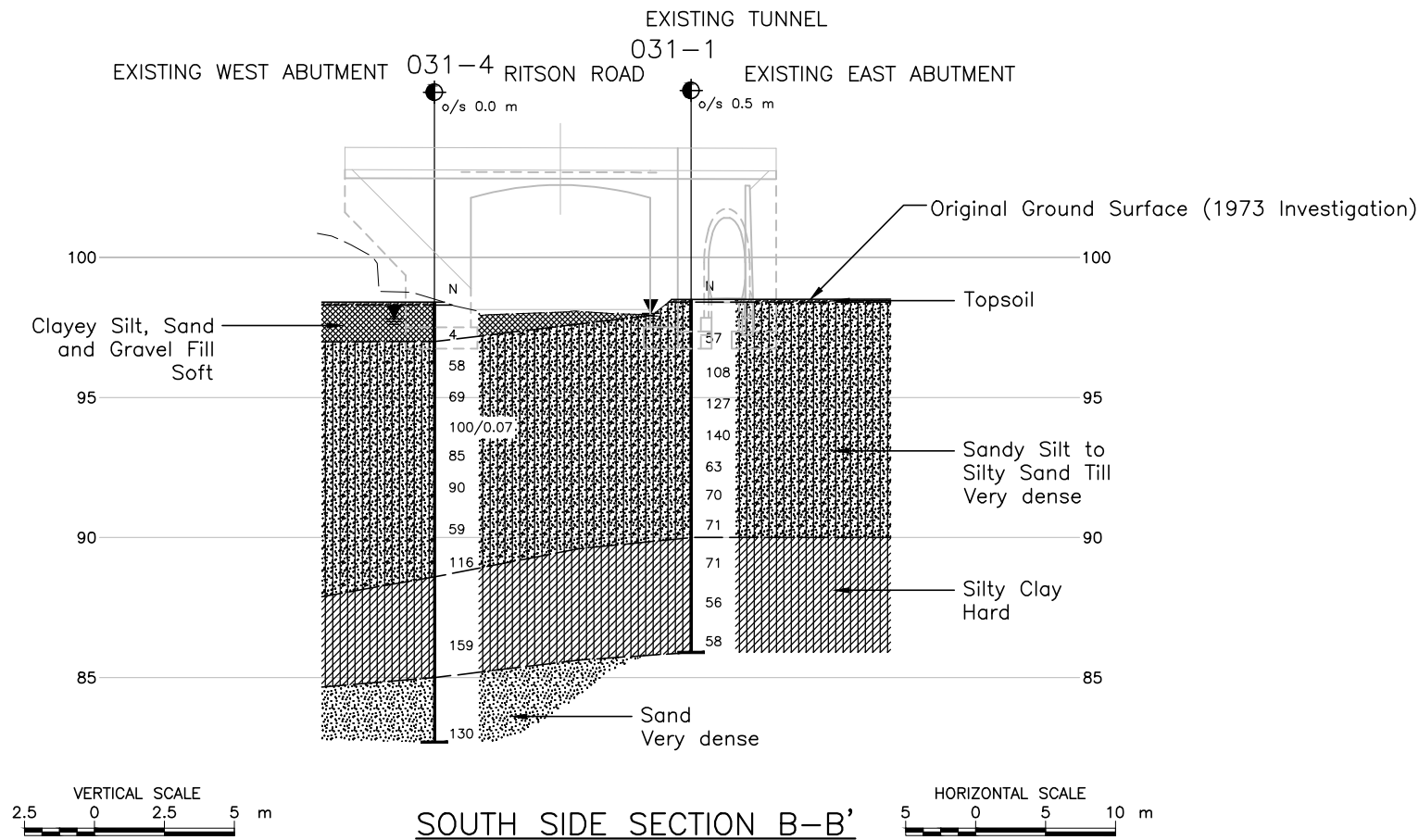
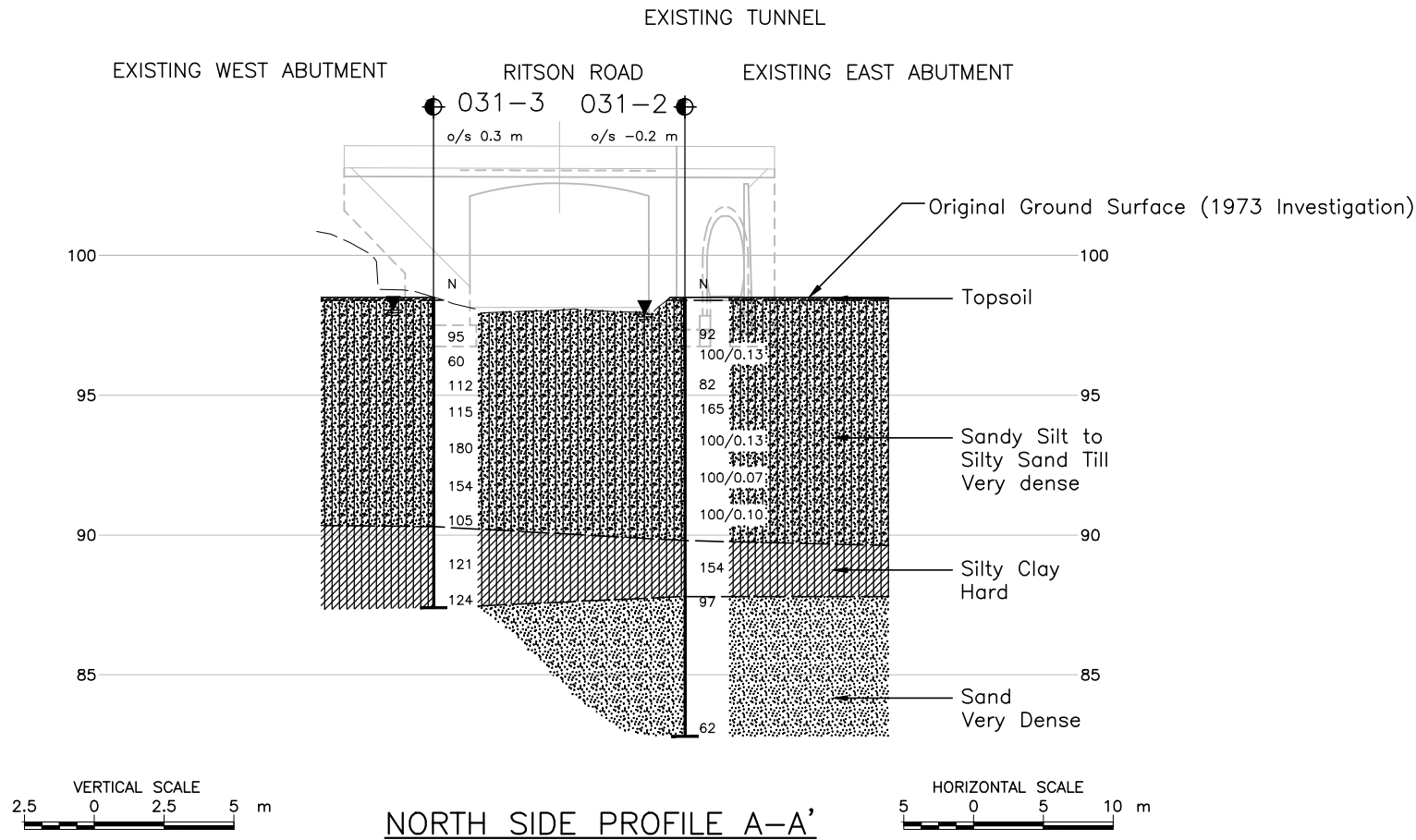
Base plans provided in digital format by URS, drawing file nos. X-Base.dwg, X-Property.dwg and Street Names.dwg, and the Proposed Design obtained from drawing file x-design\_130625.dwg, all dated July 05, 2013, received April 11, 2014. General Arrangement provided in digital format by AECOM file no. 01\_RitsonRd\_Overpass\_GA.dgn, received June 25, 2015.



NO.	DATE	BY	REVISION
Geocres No. 30M15-297			
HWY. 401		PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: 4/5/2017	SITE: 22-179
DRAWN: JFC/DD	CHKD. NK	APPD. LCC	DWG. 1





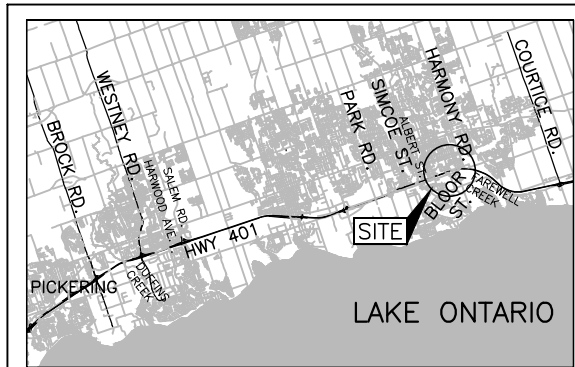


**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WO No. 10-20011

RITSON ROAD OVERPASS  
HIGHWAY 401 IMPROVEMENTS  
SOIL STRATA

SHEET



- LEGEND
- 1973 Boreholes - GEOCREs No. 30M15-031
  - Seal
  - Piezometer
  - N Standard Penetration Test Value
  - 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
  - WL in open borehole, measured 1973

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
031-1	98.5	4860607.7	357328.7
031-2	98.5	4860643.2	357316.5
031-3	98.5	4860638.1	357299.3
031-4	98.4	4860601.5	357311.4

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. 01\_RitsonRd\_Overpass\_GA.dgn, received June 25, 2015.

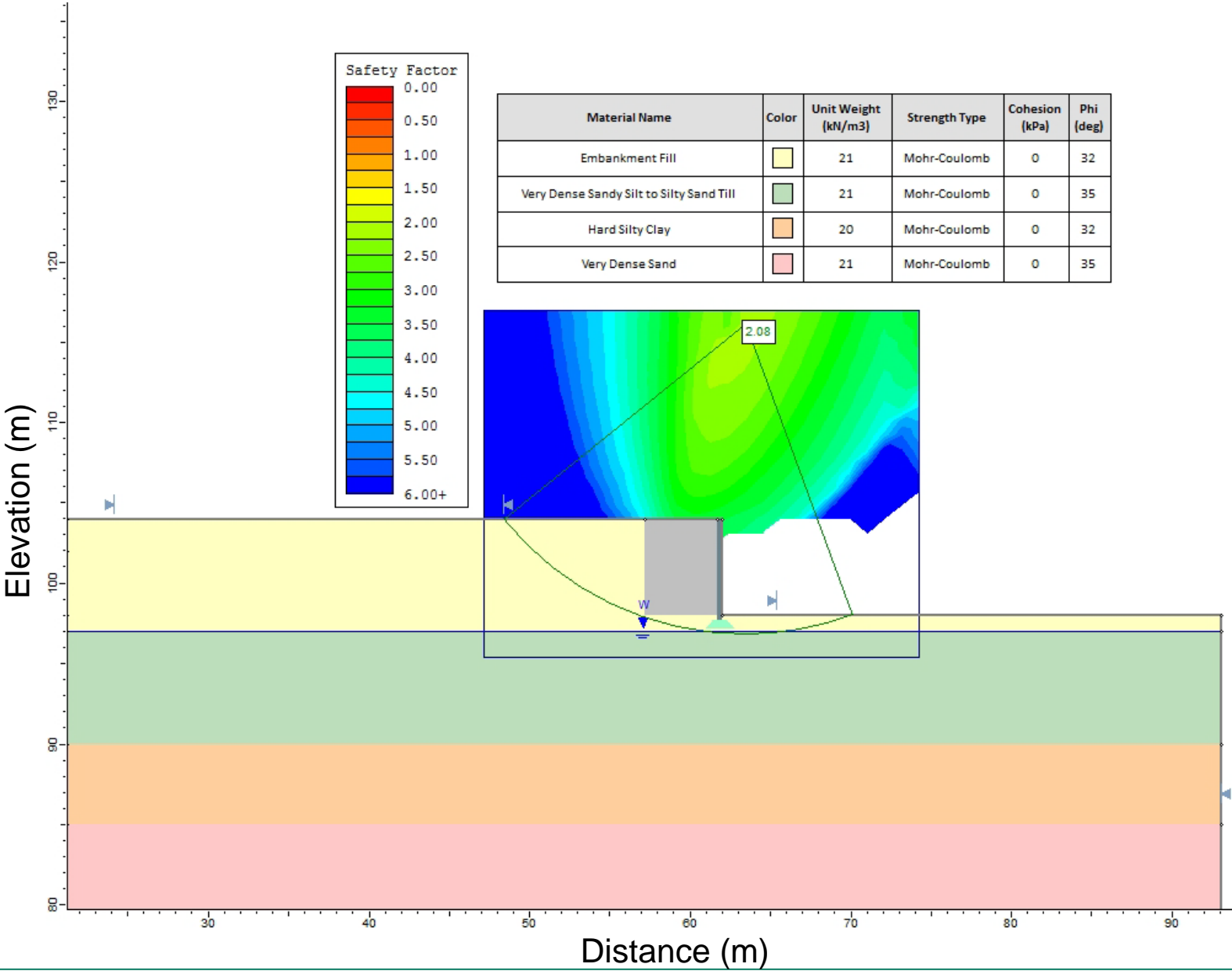


NO.	DATE	BY	REVISION
Geocres No. 30M15-297			
HWY. 401	PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. NK	CHKD. LCC	DATE: 4/5/2017	SITE: 22-179
DRAWN: DD	CHKD. NK	APPD. LCC	DWG. 2



STATIC GLOBAL STABILITY  
RITSON ROAD OVERPASS – RETAINED SOIL SYSTEM WALL

Figure 1







# **APPENDIX A**

## **Borehole Records and Laboratory Test Results 1973 Investigation (GEOCRES No. 30M15-031)**

## RECORD OF BOREHOLE № 1

JOB 73-11051

LOCATION Co-ords. 15,946,142 N; 1,172,281 E.

ORIGINATED BY CSP

W.P. 44-71-09

BORING DATE July 12, 1973

COMPILED BY CSP/

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

CHECKED BY SL

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100	LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$ $w_p$ — $w$ — $w_L$ WATER CONTENT % 10 20 30	BULK DENSITY $\gamma$ P.C.F.	REMARKS		
(#) ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE						BLOWS/FOOT	SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE
323.2	Ground Level										
	Topsoil										
	Het. mixture of sandy silt to silty sand, gravel and clay.		1	SS	57	320					
			2	SS	108						
			3	SS	127						
	(Glacial Till)		4	SS	140	310					
			5	SS	63						
	Brown-Grey		6	SS	70						
	Very Dense		7	SS	71	300					
295.2			8	SS	71	290					
28.0	Silty clay, traces of sand.		9	SS	56						
	Grey		10	SS	58						
281.7	Hard										
41.5	End of Borehole										

20  
15  $\phi$  5 % STRAIN AT FAILURE  
10

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

# RECORD OF BOREHOLE NO 2

JOB 73-11051

LOCATION Co-ords. 15,946,261 N; 1,172,246 E.

ORIGINATED BY CSP

W.P. 44-71-09

BORING DATE July 13, 1973

COMPILED BY CSP

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE				LIQUID LIMIT — W <sub>L</sub>			BULK DENSITY	REMARKS		
(ft)	ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FOOT	BLOWS / FOOT				PLASTIC LIMIT — W <sub>p</sub>					
								20	40	60	80	100	WATER CONTENT — W				
													W <sub>p</sub> — W — W <sub>L</sub>				
SHEAR STRENGTH P.S.F.							WATER CONTENT %										
○ UNCONFINED + FIELD VANE							10 20 30										
● QUICK TRIAXIAL × LAB VANE																	
5 0	323.3	Ground Level															
		Topsoil															
		Het. mix. of sandy silt to silty sand, gravel and clay.		1	SS	92	320										
				2	SS	100/5"											
				3	SS	82											
		(Glacial Till)		4	SS	165	310										
		Brown-Grey		5	SS	100/5"											
		Very Dense		6	SS	100/3"											
				7	SS	100/4"	300										
294.8	28.5	Silty clay, traces of sand.		8	SS	154	290										
288.3	35.0	Grey Hard		9	SS	97											
		Sand, traces of silt and gravel.					280										
		Grey															
		Very Dense		10	SS	62											
271.8	51.5	End of Borehole					270										

DESIGN SERVICES BRANCH

# RECORD OF BOREHOLE NO 3

FOUNDATIONS OFFICE

JOB 73-11051 LOCATION Co-nrds. 15,946,244 N; 1,172,190 E.

W.P. 44-71-09 BORING DATE July 13, 1973

DATUM Geodetic BOREHOLE TYPE Auger and Cone Test

ORIGINATED BY CSP

COMPILED BY CSP

CHECKED BY *AK*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT — $w_L$ PLASTIC LIMIT — $w_p$ WATER CONTENT — $w$			BULK DENSITY $\gamma$	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	$w_p$	$w$	$w_L$	
323.2	Ground Level														
	Topsoil														
	Het. mix. of sandy silt to silty sand, gravel and clay.		1	SS	95	320									
			2	SS	60										
			3	SS	112										
	(Glacial Till)		4	SS	115	310									
			5	SS	180										
	Brown-Grey		6	SS	154										
			7	SS	105	300									
296.2	Very Dense														
27.0	Silty clay, traces of sand.		8	SS	121	290									
	Grey														
286.7	Hard		9	SS	124										
36.5	End of Borehole					280									

(m)

98.5  
0.0

90.3  
8.2

87.4  
11.1

321.7  
(98.0m)

17 42 33 8

26 39 29 6

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

# RECORD OF BOREHOLE NO 4

JOB 73-11051

LOCATION Co-ords. 15,946,124 N; 1,172,233 E.

W.P. 44-71-09

BORING DATE July 16, 1973

DATUM Geodetic

BOREHOLE TYPE Auger and Cone Test

ORIGINATED BY CSP

COMPILED BY CSP

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 $\gamma$ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		20	40	60	80	100	$w_p$	$w$	$w_L$		
322.7	Ground Level															
318.2	Fill Material Clayey silt, sand & gr		1	SS	11	320										320.7 (97.7m)
4.5	Brown Soft		2	SS	58											
	Het. mix. of sandy silt to silty sand, gravel and clay.		3	SS	69											
	(Glacial Till)		4	SS	100	310										6 43 39 12
			5	SS	85											
	Brown-Grey		6	SS	90											
	Very Dense		7	SS	59											14 40 36 10
290.7			8	SS	116											
32.0	Silty clay, traces of sand.					290										
	Grey Hard		9	SS	159											
278.7						280										
44.0	Sand, traces of silt and gravel.															
271.2	Grey Very Dense		10	SS	130											0 94 (6)
51.5	End of Borehole					270										





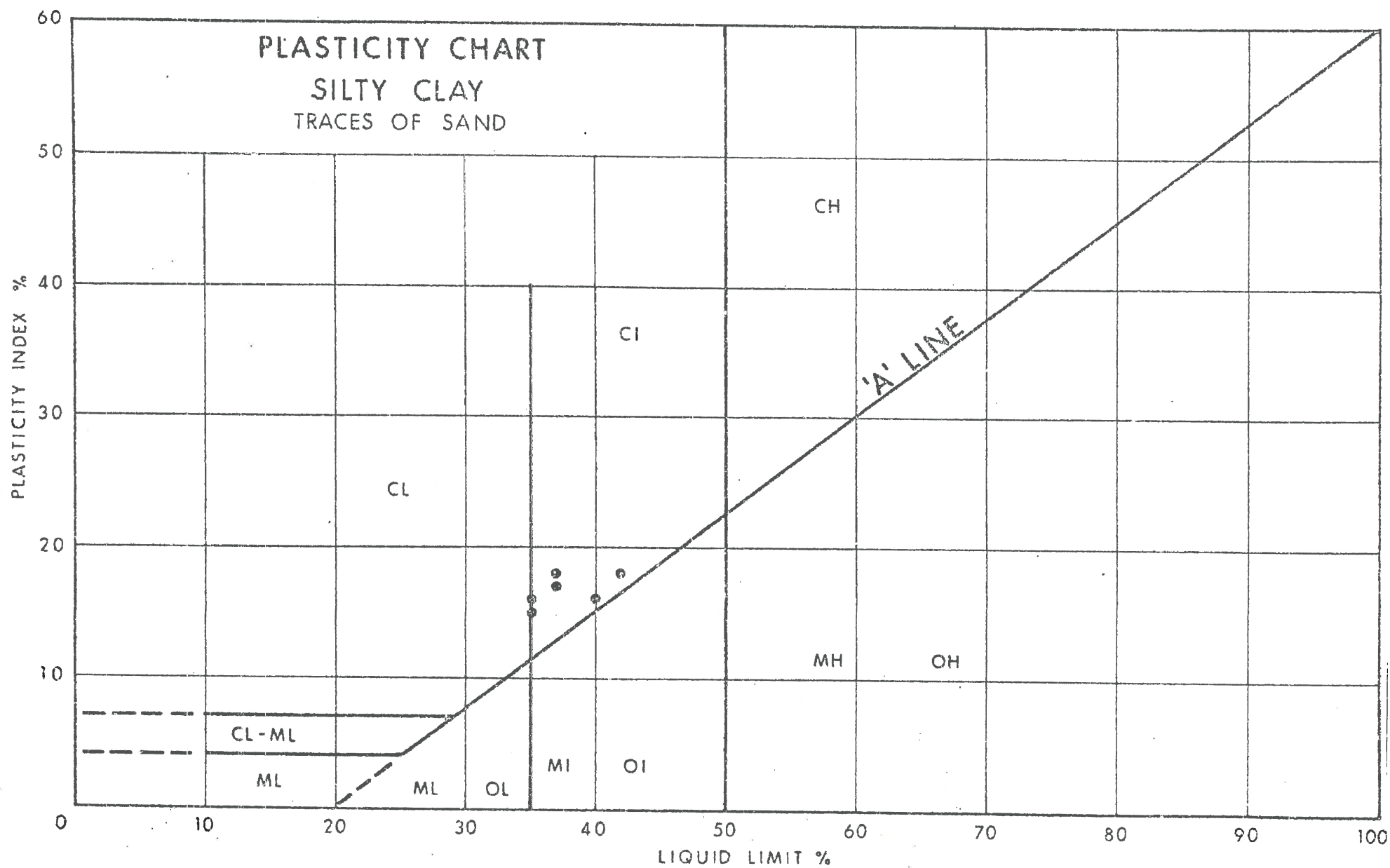
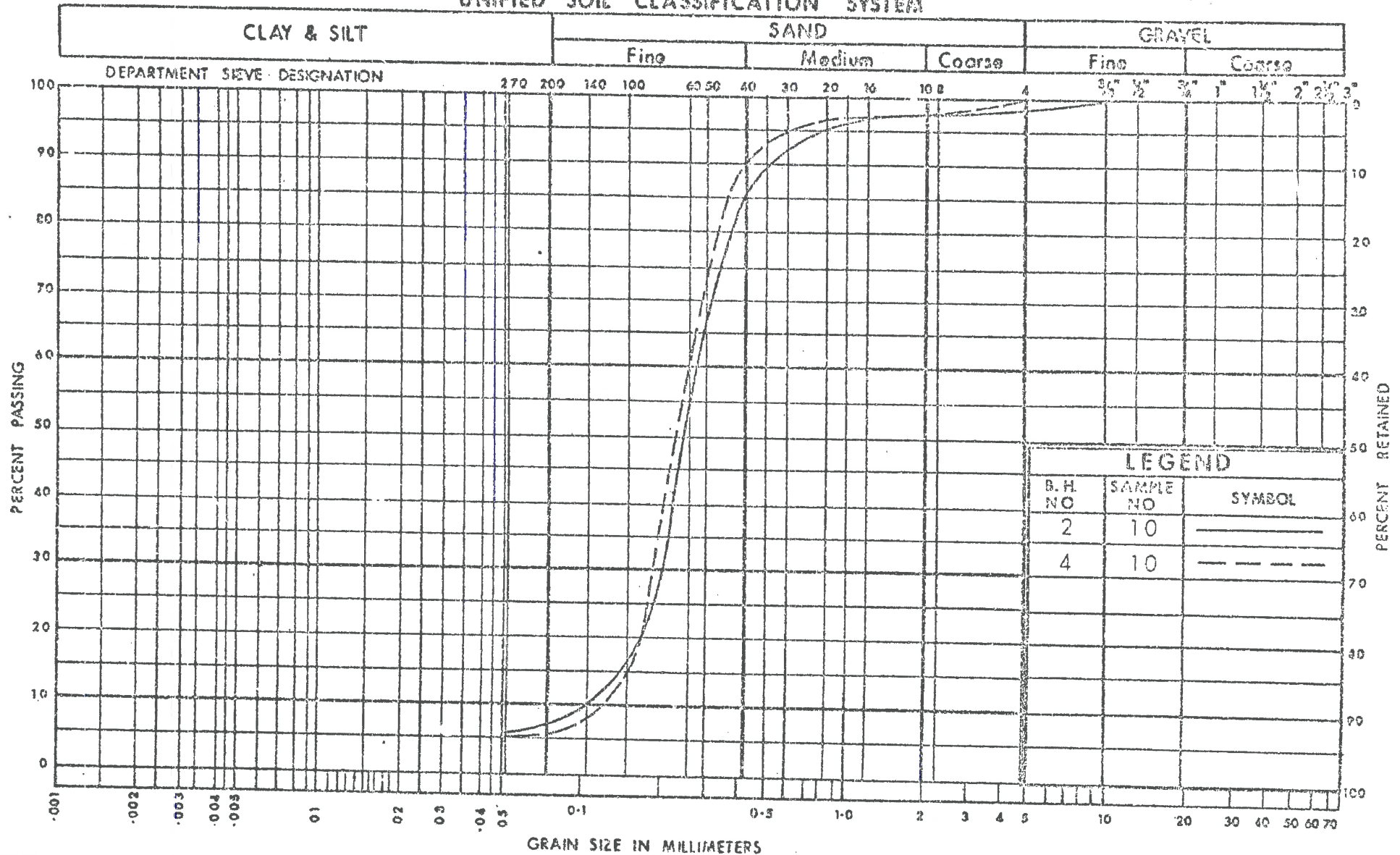


FIG. 2

W.O. 73-11051

# UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT  
OF  
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES  
BRANCH

GRAIN SIZE DISTRIBUTION  
SAND  
TRACES OF SILT & GRAVEL

W.P. No. 44-71-09

JOB No. 73-11051

FIG. 3



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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