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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Park Road Overpass Structure Site No. 22-173 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
PARK ROAD OVERPASS
STRUCTURE SITE NO. 22-173
HIGHWAY 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 and widening of the existing Park Road overpass. This report was developed with information from a previous geotechnical/foundation investigation at the Park Road overpass site, as follows:

- **MTO GEOCREs No. 30M15-7:** "Foundation Investigation Report for Proposed Overpass Structure Extension at the Crossing of Park Road and Highway 401, Township of E. Whitby, Ontario, Site No. 22-173, W.O. 72-11149, W.P. 44-71-06," prepared by MTO Foundations Office, dated March 1973.

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 Park Road overpass is located approximately 0.6 km east of Stevenson Road and 1 km west of Simcoe Street in the City of Oshawa, in the Regional Municipality of Durham (see key plan on Drawing 1). The existing Park Road overpass consists of a single-span concrete rigid frame structure supported on spread footings.

The overall surface topography in the vicinity of the site is generally flat-lying, with the ground surface rising to the north. Park Road has been constructed near the original ground surface in the area, with the existing Park Road grade at about Elevation 106 m under the structures. Highway 401 has been constructed on embankment fill that is up to approximately 5 m in height, with the pavement grade at approximately Elevation 111 m. A pedestrian tunnel runs under Highway 401 immediately east of the structure, with a retaining wall extending along the east side of the sidewalk to retain the Highway 401 embankment fill.

A residential area is present in the northeast quadrant of the structure site, and commercial development is present in the southeast quadrant, as well as along the south side of Bloor Street south of the site. Undeveloped greenspace is present in the immediate northwest and southwest quadrants of the structure site.

3.0 INVESTIGATION PROCEDURES

Four boreholes were advanced at this site as part of a previous geotechnical/foundation investigation by MTO in January 1973, in support of the widening of Highway 401 and the Park Road overpass at that time (MTO GEOCREs Report No. 30M15-7). 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 GEOCREs report.



The boreholes were advanced with a conventional diamond drill rig, adapted for soil sampling purposes, using BW casing. The boreholes extended to depths ranging from 7.2 m to 16.4 m. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers driven by a manual hammer in accordance with the Standard Penetration Test (SPT) procedure. The water level in the open boreholes was observed during the drilling operations.

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 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 the interpreted stratigraphic profiles are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

The boreholes from the 1973 investigation did not penetrate the Highway 401 embankment fill, nor the fill within the local roadway. In general, based on the 1973 investigation results, the site is underlain by a compact to very dense/hard glacial till deposit that generally grades from non-cohesive sand/silt till, to cohesive clayey silt till with depth. A layer of sand and gravel was encountered below the till at the base of Borehole 4 in the northwest quadrant of the site. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Sand and Silt Till to Clayey Silt Till

A till deposit was encountered below the ground surface at the time of the 1973 investigation in all four boreholes. The surface of the till was encountered between approximately Elevation 106.0 m and 107.8 m. Boreholes 1 to 3 were terminated within this deposit after penetrating it for a thickness of approximately 7.2 m to 16.4 m; Borehole 4 extended 6.6 m through this deposit (or a portion thereof) before encountering a layer of sand and gravel.

The upper 3.4 m to 7.2 m portion of the till (extending to approximately Elevation 102.6 m to 99.2 m) is described as non-cohesive, consisting of sand and silt containing trace to some gravel and trace clay. Below this in



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Boreholes 2 and 3, the deposit grades to a cohesive till, consisting of clayey silt with sand to some sand, containing trace to some gravel. The results of the grain size distribution tests completed on selected samples of the till deposit are shown on the boreholes records and on Figure 2 contained in Appendix A. The till deposit is glacially derived, and therefore should be expected to contain cobbles and boulders, although no such obstructions are noted on the borehole records.

Atterberg limits tests were carried out on selected sample of the till deposit and measured plastic limits of approximately 10 to 11 per cent, liquid limits of approximately 11 to 18 per cent, and plasticity indices of approximately 2 to 8 per cent. These results, which are shown on a plasticity chart on Figure 1 in Appendix A, confirm that the upper portion of the till deposit is non-plastic to slightly plastic, while the lower portion of the till deposit is a clayey silt of low plasticity.

The measured Standard Penetration Test (SPT) 'N'-values recorded within the till deposit are typically greater than 40 blows per 0.3 m of penetration, with many SPT 'N'-values greater than 100 blows per 0.3 m of penetration, indicative of a dense to very dense relative density or hard consistency. However, an approximately 8 m thick zone of less stiff clayey silt till was encountered in Boreholes 2 and 3, on the south side of the highway, between about Elevation 102 m and 94 m; within this zone, the SPT 'N'-values range from 15 to 39 blows per 0.3 m of penetration, but are typically less than 30 blows, suggesting that this portion of the clayey silt till has a very stiff consistency.

4.2.2 Sand and Gravel Layer

A sand and gravel layer was encountered at the base of Borehole 4. The borehole terminated within this deposit, penetrating it for a thickness of 2.7 m, between approximately Elevation 101.2 m and 98.5 m.

The measured SPT 'N'-values recorded within the sand and gravel deposit were greater than 100 blows per 0.3 m of penetration, indicating a very dense relative density.

4.3 Groundwater Conditions

The groundwater level was noted 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)	Ground Water Elevation in Open Borehole (m)	Comments
1	106.4	0.1	106.3	During drilling – January 1973
2	106.0	0.9	105.1	During drilling – January 1973
3	106.0	1.3	104.7	During drilling – January 1973
4	107.8	2.3	105.5	During drilling – January 1973

Based on these measurements, the groundwater level at the site during the 1973 investigation is relatively shallow, between about Elevation 104.7 m and 106.3 m. However, the water levels observed in the open boreholes during



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the investigation may not represent the long-term stabilized groundwater levels, nor the current groundwater regime at the site. Further, the water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Patrick Speirs and reviewed by Ms. Nikol Kochmanova, P.Eng. 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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**PRELIMINARY FOUNDATION REPORT
PARK ROAD OVERPASS, W.O. 10-20011**

PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
PARK ROAD OVERPASS
STRUCTURE SITE NO. 22-173
HIGHWAY IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
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6.0 DISCUSSION AND PRELIMINARY FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation engineering recommendations for the proposed replacement and widening of the existing Park Road overpass and associated wingwalls/retaining walls. These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a 1973 geotechnical 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 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 Park Road overpass will be replaced with a new single-span overpass structure that is both longer and wider.

The existing overpass was constructed in 1955, and widened by about 6 m on both the south and north sides in 1975. Based on the available design drawings (WP No. 44-71-06, Sheets 1, 3 and 4: "Park Road Overpass Widening", prepared by the Ministry of Transportation and Communications Ontario, dated June 1975), the existing structures are supported on spread footings that are founded at approximately Elevation 104.6 m, about 1.2 m to 1.4 m below the current Park Road grade.

Based on the preliminary General Arrangement (GA) drawing provided by AECOM, Park Road is proposed to be widened from two to four lanes plus pedestrian sidewalks on both sides, resulting in a new span length of about 24.7 m. In order to accommodate the widening of Highway 401, the new overpass will be wider by about 12 m on the north side and about 13 m on the south side. New retaining walls will be required parallel to Park Road along the west side of the road and in the northeast quadrant; in the southeast quadrant, a new retaining wall will be required parallel to Highway 401. In conjunction with the overpass reconstruction, Park Road will be lowered by approximately 1.5 m, such that its new grade will be at about Elevation 104.5 m below the highway.

Both shallow and deep foundation options have been considered for support of the new Park 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 sand and silt till deposit:** Shallow footings are feasible at this site due to the generally very dense nature of the overburden 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 Park Road grade (at approximately Elevation 104.5 m, about 1.5 m below the current local road grade). Temporary protection systems will be required along Highway 401, as well as at the Park 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 would extend below the groundwater level at the site, and some groundwater control is expected to be required to enable shallow foundations to be constructed in “dry” conditions within the non-cohesive till.

- **Footings “perched” on a compacted granular pad in the approach embankment:** At the abutments, footings “perched” in the approach embankments above the Park 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 tube 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 the new Park Road grade (but not to frost depth below this level, as would be the case for spread footings). Very dense/hard “100-blow” soil will be encountered at shallow depths below the Park 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 retaining walls for the new overpass, spread/strip footings should be founded on the very dense sand and silt till deposit; concrete retaining walls may also be founded on compacted granular pads that are stepped up into the approach embankments, although this configuration may not be applicable for this site based on the proposed wall geometry, which is predominantly parallel to Park Road. Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest



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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.

Park Road is presently at about Elevation 106 m; however, its grade is proposed to be lowered to approximately Elevation 104.5 m below the overpass. As such, the maximum (highest) founding elevation recommended for the preliminary design of the abutment and wall footings is approximately Elevation 103.3 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 silt and sand to clayey silt till deposit (for the abutment footings, and for retaining wall footings founded below the Park Road grade), or on a compacted Granular 'A' pad having a minimum thickness of 1 m (for retaining wall footings, where these may be stepped upward into the approach embankments, provided they are at least 1 m above the Park Road grade).

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 sand and silt till or very stiff to hard clayey silt till	500	350
Wall footing on minimum 1 m thick Granular 'A' pad	750	350

Note: 1. The geotechnical resistance/ reaction values given above are estimated for a 3 m wide spread/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" sand and silt to clayey silt till deposit. 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 Park Road approach embankments, with the underside of the new pile caps at approximately Elevation 108.5 m. The following pile tip elevations are recommended for preliminary design.



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Foundation Element	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)
WBL Structure – West and East Abutments	100.5 – 101.5	97
EBL Structure – West and East Abutments	93	91

Based on the above elevations, the proposed piles are estimated to be approximately 11.5 m to 17.5 m long. Given the relatively shallow depth to the upper layer of very dense ("100-blow") sand and silt till, pre-augering may be required.

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 OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 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 lower "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:



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Foundation Element	Approximate Surface Elevation of "100-Blow" Soil (m)	Estimated Design Tip Elevation (m)
WBL Structure – West and East Abutments	100.5 – 101.5	97
EBL Structure – West and East Abutments	93	91

The caissons will extend into and through water-bearing sand and silt till; in addition, a sand and gravel deposit or layer was encountered in one of the boreholes; the vertical and lateral extent of this deposit or layer is unknown, but similar layers may be present elsewhere within the till deposit. If caisson foundations are adopted, a temporary liner and/or drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures to minimize disturbance to the side walls and 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:

Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
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 any 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 below the Park Road grade, and below the groundwater table, as compared to strip footings. Retaining walls will be required parallel to Park Road, except within the southeast quadrant of the structure site where the retaining wall is proposed to be parallel to Highway 401.

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.

Assuming that the RSS wall (estimated to be up to approximately 6.5 m high) 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 any additional subsurface information at that time.

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):

Soil Deposit	Bulk Unit Weight (kN/m³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	32°	-
Upper "100-blow" glacial till	21	35°	-
Very stiff clayey silt till	21	30°	-
Lower glacial till	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 6.5 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 the retaining 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.

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



6.6.4 Settlement

At this preliminary stage, it is estimated that for widened approach embankments on Highway 401, the settlement of the underlying soils will be less than about 25 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 Approach Embankments

The existing Highway 401 approach embankments are approximately 5 m to 5.5 m high relative to the natural ground surface; with a nominal grade raise on Highway 401 and the proposed lowering of Park Road, the approach embankments will have a maximum height of approximately 7 m relative to the proposed lowered Park Road grade. Based on the GA drawing provided by AECOM, the approach embankments are proposed to be widened by approximately 3 m to 5 m on the south side, and up to about 14 m on the north side.

6.7.1 Subgrade Preparation and Embankment Construction

It is recommended that any topsoil and surficial soft/loose soils should be stripped from the footprint of the embankment widening. 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.

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.

6.7.2 Slope Stability

Preliminary static slope stability analysis have been completed for the Highway 401 approach embankment, using the commercially available program *Slide 6.0*, from Rocscience, to check that the target minimum factor of safety is achieved. A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed slope widening on this project, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were completed for an overall 7 m high slope in long-term (effective stress) conditions, using the parameters outlined in Section 6.6.3. The results of the static global stability analysis indicate that a minimum factor of safety of 1.3 is achieved for 7 m high slopes, oriented no steeper than 2H:1V, as shown on Figure 2. This preliminary assessment of the slope stability of the approach embankments should be reviewed and confirmed during detailed design based on additional subsurface information as may be available.

6.7.3 Settlement under Widened Embankment Loading

Preliminary settlement assessments have been completed for the proposed Highway 401 embankment widening using the commercially available computer program *Settle-3D 2.0* from Rocscience, using the consolidation parameters and estimated elastic deformation moduli given in the table below, based on the results from 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).



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

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)
Embankment fill	21	-
Compact to very dense sand and silt till (upper portion of till deposit)	21	100 MPa
Very stiff to hard clayey silt till	21	35 MPa
Very dense/hard clayey silt till (lower portion of till deposit)	21	150 MPa

Based on this preliminary assessment, the settlement of the foundation soils under the approximately 3 m to 5 m widening on the south side is estimated to be approximately 10 mm under the 7 m high approach; settlement of the foundation soils under the approximately 14 m widening on the north side is estimated to be approximately 25 mm under the 7 m high approach. The majority of this settlement is estimated to be completed during and immediately following completion of the embankment widening.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.8 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.

6.8.1 Open-Cut Excavation and Temporary Protection Systems

The construction of the spread/strip footings would require excavations up to about 3 m below the existing Park Road grade. If integral abutment foundations are adopted, excavation will still be required to at least 1.5 m below the existing Park Road grade (i.e., to near or slightly below the new Park 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 are expected to extend below the groundwater table. The existing fill is 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 Park 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.

6.8.2 Groundwater Control

The groundwater level was measured in the 1973 investigation between Elevation 104.6 m to 106.3 m, near or just below the existing Park Road grade. With the proposed lowering of the Park Road grade to approximately Elevation 104.5 m, 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. However, additional borehole investigation is recommended to confirm the 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, as the flow rate would be dependent on whether the contractor includes in 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 m³/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 generally 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.

6.8.3 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.

6.8.4 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.

6.8.5 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.



6.9 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 pile tip elevations;
- The subsurface conditions within the footprint of the retaining walls, and within the footprint of the embankment widening areas, to confirm the geotechnical resistances, settlement assessment and global stability; and
- 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 Kochmanova, P.Eng.
Geotechnical Engineer



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

PKS/NK/LCC/sm

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

REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition, McGraw Hill Book Company, New York.
- 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*, 3rd Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society, BiTech Publisher Ltd., British Columbia.
- Canadian Standards Association (CSA), 2006. *Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6 06*. CSA Special Publication, S6.1 06.
- Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd 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 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

Ontario Occupational Health and Safety Act:

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

TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES – PARK ROAD OVERPASS

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Spread/strip footings	<ul style="list-style-type: none"> Feasible for support of the new abutments, and for support of concrete retaining walls 	<ul style="list-style-type: none"> Conventional excavation and construction techniques 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 	<ul style="list-style-type: none"> Excavation will extend below the groundwater level and groundwater control will be required Significant temporary protection systems required through Highway 401, with shorter protection systems likely required parallel to Park Road 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations 	<ul style="list-style-type: none"> Risk of softening/ loosening of footing subgrade; groundwater control is critical
Driven steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of abutments Not required for support of retaining walls 	<ul style="list-style-type: none"> Conventional construction methods for H-pile or steel pipe pile foundations Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration 	<ul style="list-style-type: none"> 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) Due to the shallow depth to “100-blow” material, pre-augering will likely be required 	<ul style="list-style-type: none"> Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving



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

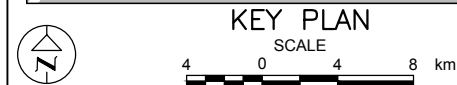
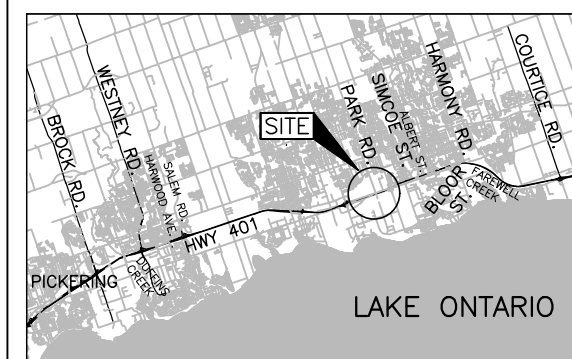
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none">• Feasible but not recommended for support of abutments	<ul style="list-style-type: none">• Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation, dewatering and protection system requirements compared with footing option• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles	<ul style="list-style-type: none">• 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• 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• Precludes use of integral abutments	<ul style="list-style-type: none">• Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary liners	<ul style="list-style-type: none">• Risk of loosening or disturbing founding soils at base of caissons

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

CONT No.
WO No. 10-20011

PARK ROAD OVERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND
SOIL STRATA

SHEET



LEGEND

- Borehole - 1973 Investigation (Geocres No. 30M15-7)
- Standard Penetration Test Value
- Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling (Jan. 1973)

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
7-1	106.4	4860071.4	355525.6
7-2	106.0	4860015.6	355527.3
7-3	106.0	4860018.5	355544.2
7-4	107.8	4860066.5	355503.3



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.

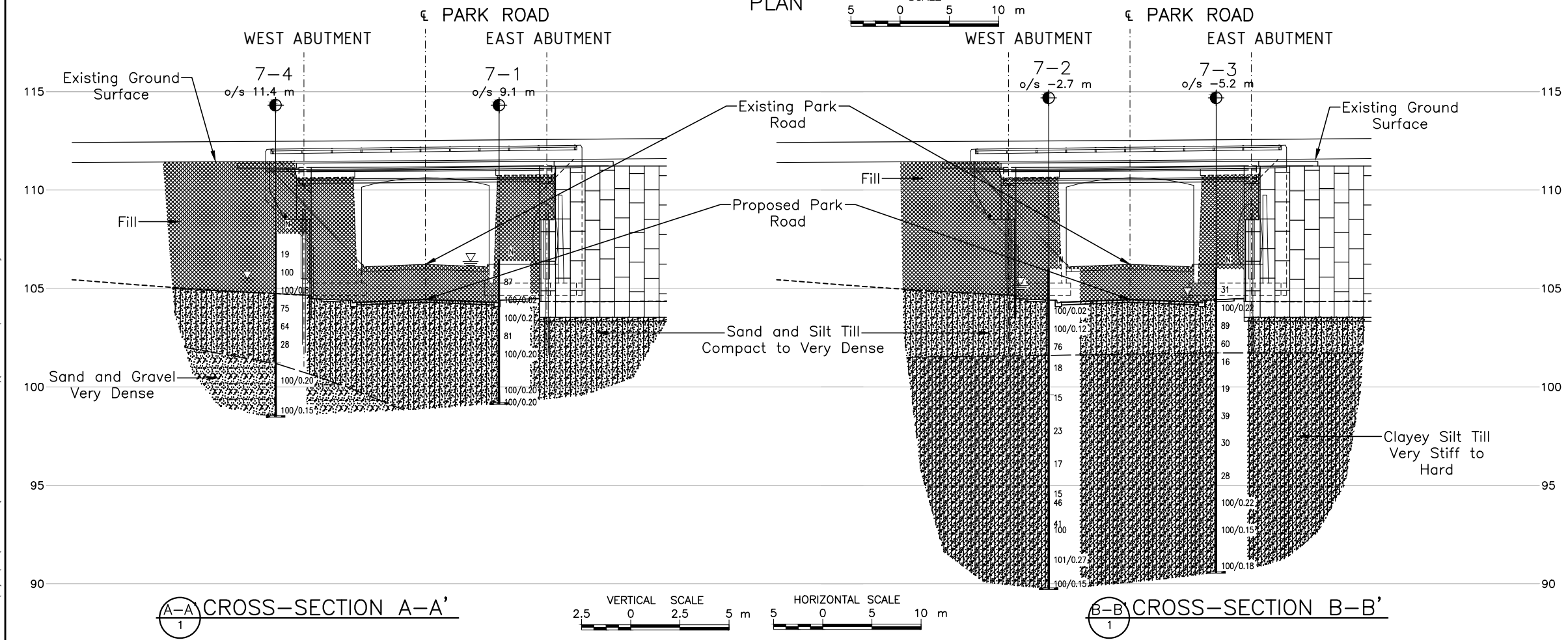
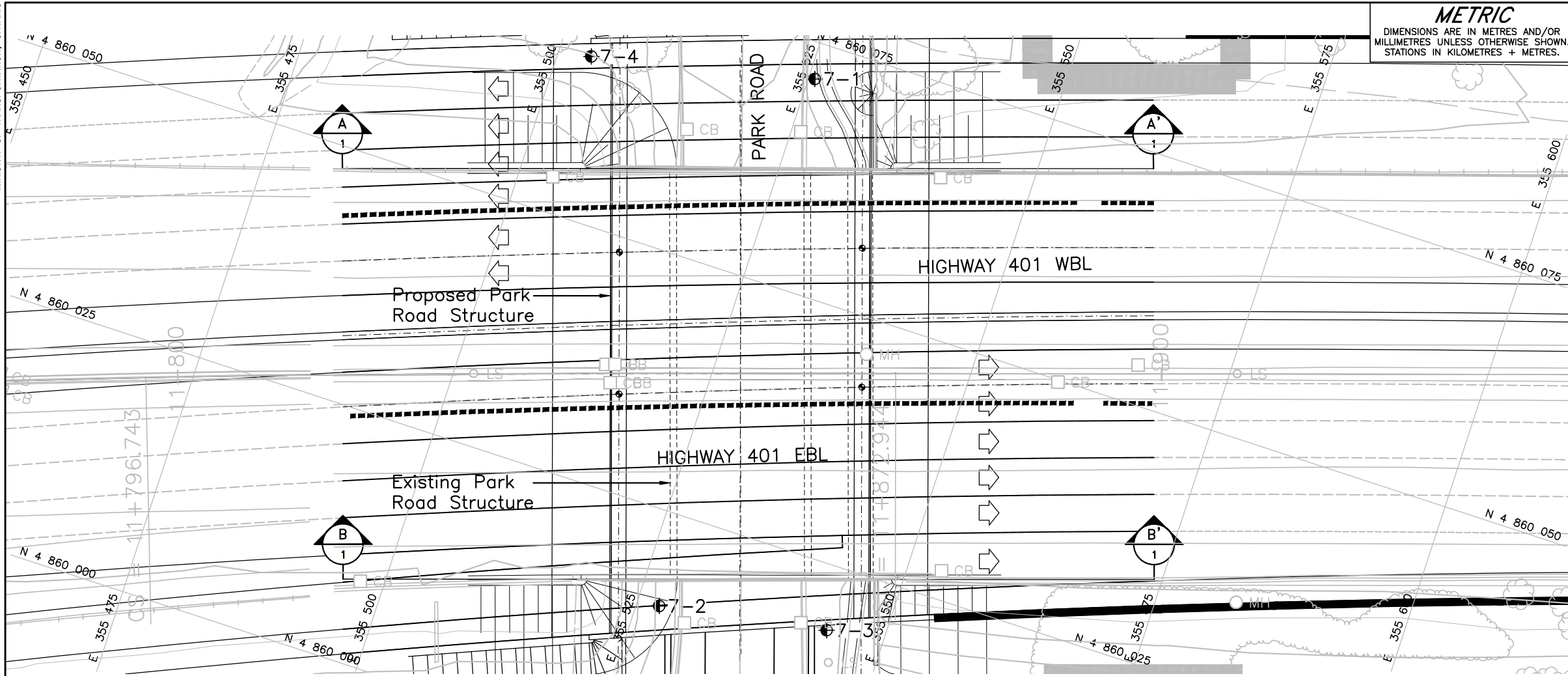
REFERENCE

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, drawing file ACAD-01_GA_Park Rd. Overpass.dwg, received July 27, 2016.

NO.	DATE	BY	REVISION
1	4/5/2017	MR	1

Geocres No. 30M15-296

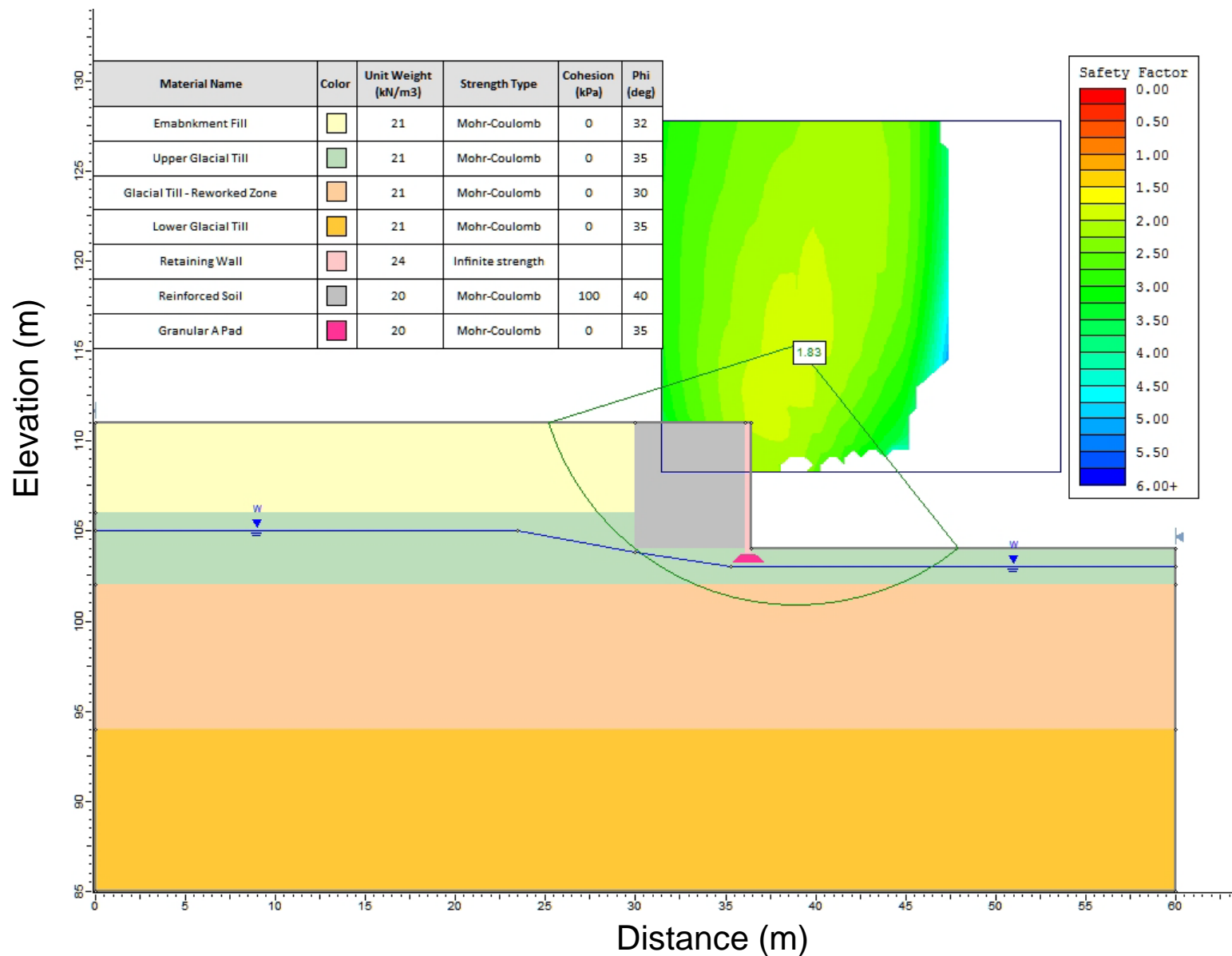
HWY. 401	PROJECT NO. 11-1184-0143	DIST. CENTRAL
SUBM'D. NK	CHKD. NK	DATE: 4/5/2017
DRAWN: MR	CHKD. NK	APPD. LCC
		SITE: 22-173
		DWG. 1





STATIC GLOBAL STABILITY PARK ROAD OVERPASS – RETAINED SOIL SYSTEM WALLS

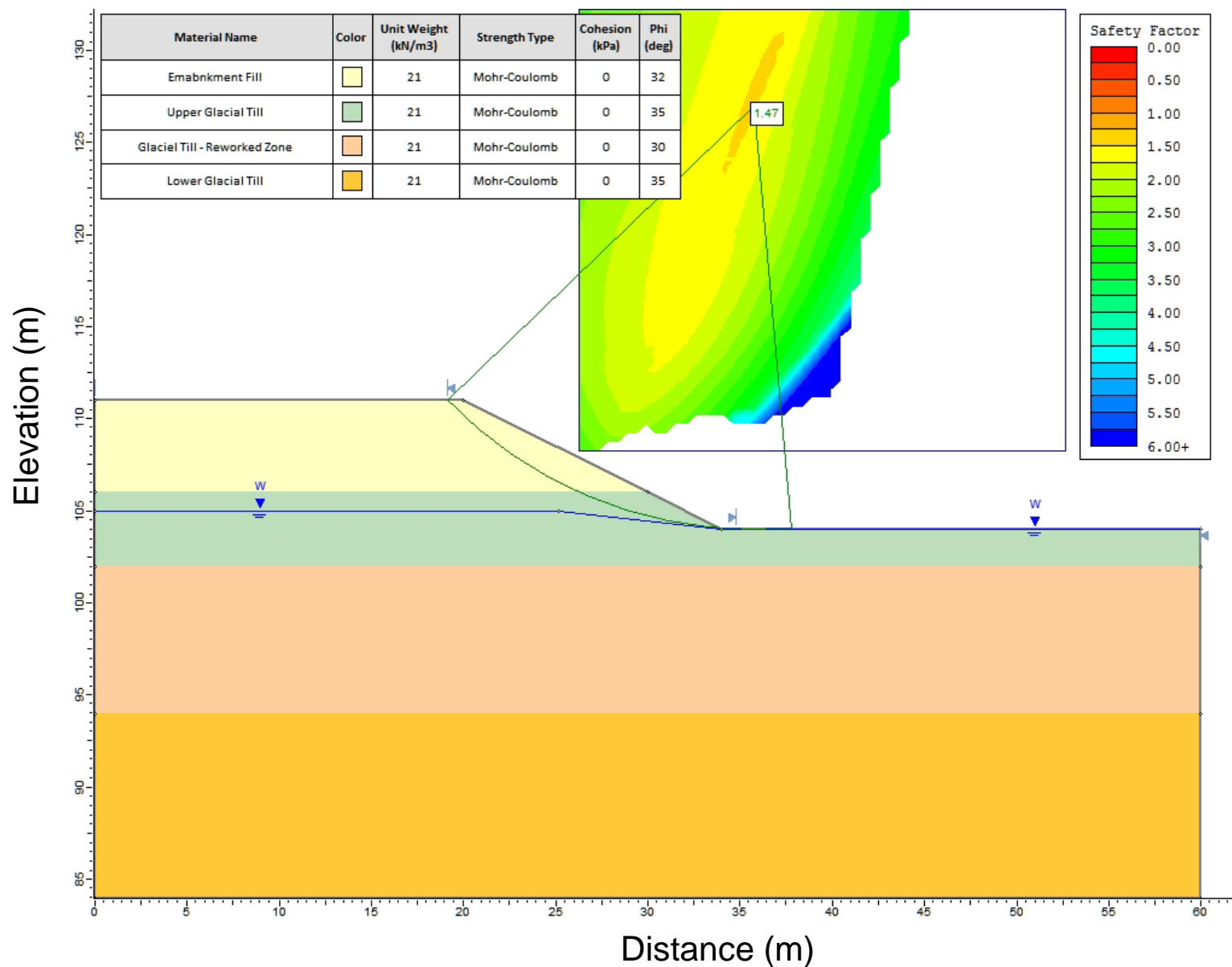
Figure 1





STATIC GLOBAL STABILITY PARK ROAD OVERPASS - APPROACH EMBANKMENTS

Figure 2





APPENDIX A

Borehole Records and Laboratory Test Results 1973 Investigation (GEOCRES No. 30M15-7)

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11119

LOCATION Co-ordinates 944,345 N. - 166,376 E.

ORIGINATED BY J.B.

W.P. 44-71-06


BORING DATE January 22, 1973

COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT _____ PLASTIC LIMIT _____ WATER CONTENT _____		BULK DENSITY	REMARKS
ELEV. DEPTH (ft)	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD V.A. ● QUICK TRIAXIAL x LAB V.A.		WATER CONTENT % 5 10 15			
349.1	Ground Level											
0.0	Hetrogeneous mixture of Sand, Silt, Gravel and Clay Brown to Grey		1	SS	87	34.0						269.0 10 20 30 40
341.1			2	SS	100							
8.0			3	SS	100							
	Grey Very Dense		4	SS	81							
			5	SS	100							
	Glacial Till		6	SS	100		33.0					
525.3			7	SS	100							
23.8	End of Borehole											

20
15
10
5
% STRAIN AT FAILURE

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE NO 2

FOUNDATIONS OFFICE

JOB NO. 23719

LOCATION Cochrane, Ont. 185 N. - 186, 187 E.

ORIGINATED BY J.H.

DATE 11-11-73

BORING DATE January 23, 1973

COMPILED BY J.H.

DRILLUM Hand-drawn

BOREHOLE TYPE B.W. Casing

CHECKED BY J.H.

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	SILICAR STRENGTH P.S.F. UNCONFINED + FIELD VANE QUICK TRIAXIAL + FIELD VANE	WATER CONTENT % 5 10 15	BULK DENSITY	REMARKS
DEPTH (m)	DESCRIPTION	STRAT. UNIT	NUMBER TYPE							
0.0	Ground Level									
3.7	Heterogeneous Mixture of Silt, Sand, Gravel and Clay.		1 SS 100/4"	310						Elev. 310.8
3.6	Very Dense, Glacial Till		2 SS 100/4"							3 43 44 10
11.0	Down to Gray		3 SS 100/4"							
	Gray		4 SS 170							
			5 SS 118	330						9 37 39 20
	Heterogeneous Mixture of Clayey Silt, Sand and Gravel.		6 SS 15							
	Very Stiff to Hard		7 SS 23	320						
	Glacial Till		8 SS 117							24 36 30 10
	Remoulded Zone Between Elev. 311-332		9 SS 115	310						
			10 SS 16							
			11a SS 111							
			11b SS 100	300						
			12 SS 101/11"							5 38 39 18
29.1	End of borehole		13 SS 100/4"	290						

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 3

JOB 72-11119

LOCATION Co-ordinates 944,194 E. - 166,437 E.

ORIGINATED BY J.B.

W.P. 44-7-06

BORING DATE January 25, 1973

COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

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 (ft)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.				w_p — w — w_L WATER CONTENT %				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				5 10 15				
347.9	Ground Level														GR.SA.SI.CL
0.0	Hetrogeneous Mixture of Silt, Sand, Gravel and Clay.		1	SS	31	340							343.4	9 36 41 14	
	Dense to Very Dense		2	SS	100/9"										
	Glacial Till		3	SS	89										
333.9	Brown to Grey		4	SS	60										4 44 38 14
14.0	Grey		5	SS	16	330									
	Hetrogeneous Mixture of Clayey Silt, Sand and Gravel		6	SS	19										
	Very Stiff to Hard		7	SS	39	320									14 41 35 10
	Re-worked Zone Between Elev. 308-334		8	SS	30										
	Glacial Till		9	SS	28	310									
			10	SS	100/9"										
			11	SS	100/6"	300									1 40 37 22
			12	SS	100/7"										
297.3															
50.6	End of Borehole					290									

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 72-11149

LOCATION Co-ordinates 944,351 N. - 166,303 E.

ORIGINATED BY J.B.

W.P. 44-76-01

BORING DATE January 26, 1973

COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY J.B.

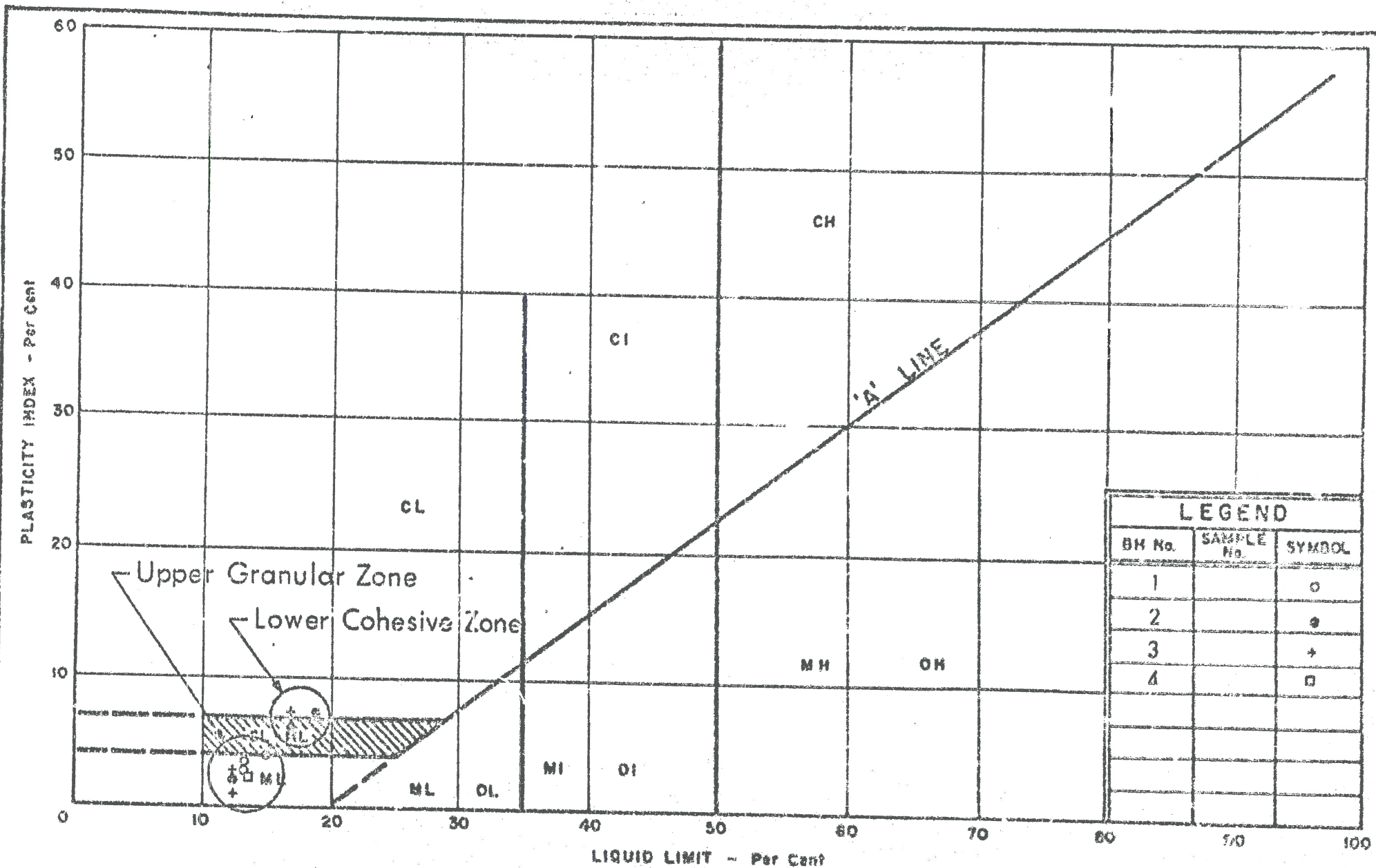
SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH (4)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.			WATER CONTENT %					
							<input type="radio"/> UNCONFINED	<input type="radio"/> QUICK TRIAXIAL	<input type="radio"/> FIELD VANE	<input type="radio"/> LAB VANE	w_p	w			w_L
353.8	Ground Level														
0.0	Hetrogeneous Mixture of Sand, Silt, Gravel and Clay Compact to Very Dense Brown to Grey		1	SS	19	350								17 37 34 12	
			2	SS	100										
			3	SS	100	8"									
	Grey		4	SS	75	340					0	1			
			5	SS	74										8 40 37 15
	Glacial Till		6	SS	28										
332.0															
21.8	Sand and Gravel		7	SS	100	8"	330								
323.3	Very Dense		8	SS	100	6"								50 47 (3)	
30.5	End of Borehole					320									

(m)
107.8
0.0

101.2
6.6

98.5
9.3

OFFICE REPORT ON SOIL EXPLORATION



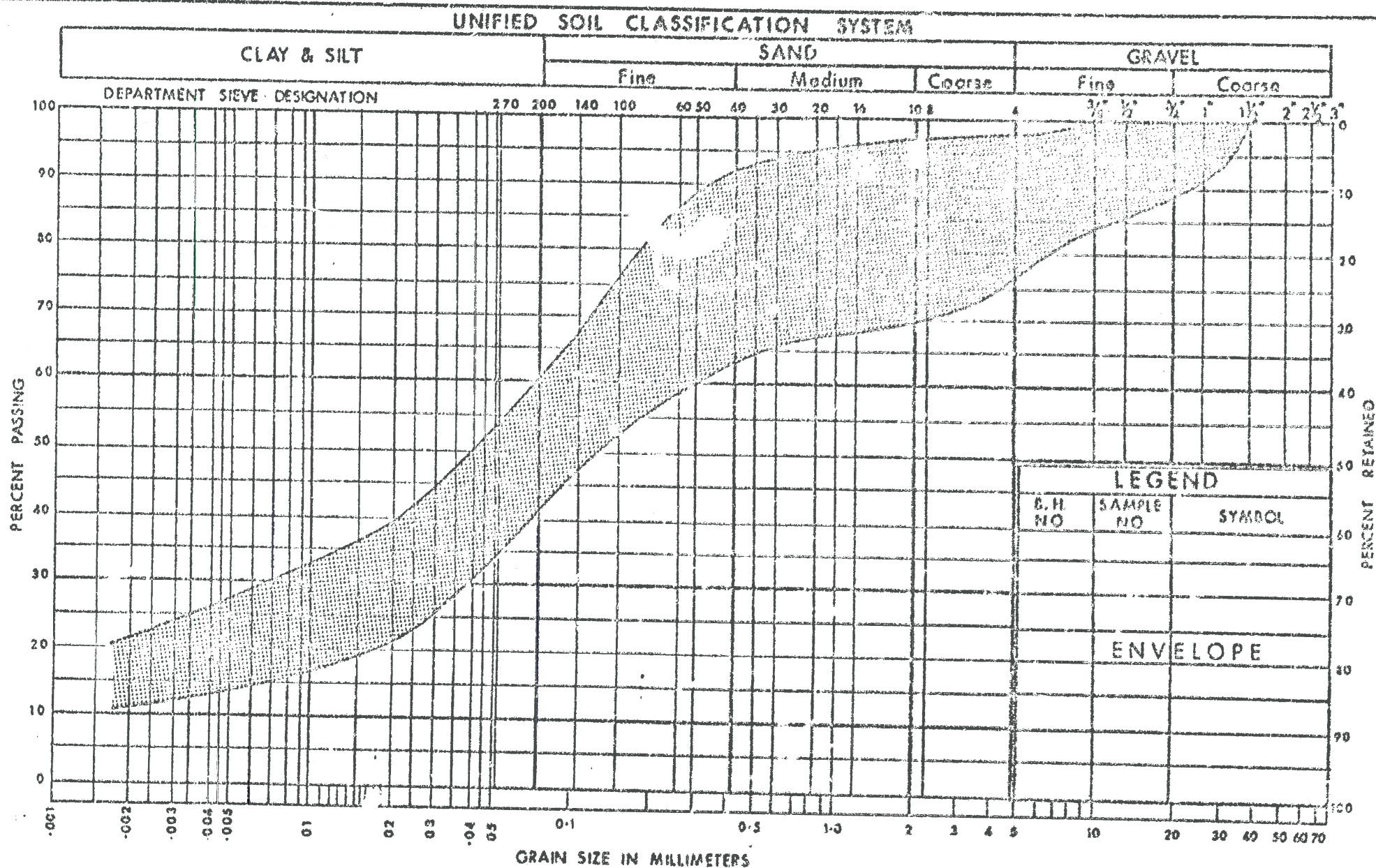
DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

PLASTICITY CHART GLACIAL TILL

WSP No. 44-71-06

JOB No. 72-11149

FIG. No. 1



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.F. No. 44-71-06

JOB No. 72-11149

FIG. N° 2

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